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Technical Basis for Flood Protection at Nuclear Power Plants

James R. Leech, Loren L. Wehmeyer, David A. Margo,
Landris T. Lee, Aaron R. Byrd, and Donald L. Ward

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Coastal and Hydraulics Laboratory
U.S. Army Engineer Research and Development Center
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

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U.S. Army Engineer Research and Development Center
3909 Halls Ferry Road, Vicksburg, MS 39180-6199

Abstract

Current flood-protection regulatory guidance for nuclear power plants is contained in the Regulatory Guide 1.102, Flood Protection for Nuclear Power Plants (Nuclear Regulatory Commission 1976). The Guide requires structures, systems, and components to withstand natural phenomena without safety-function loss. This report describes 2013 flood-protection-feature types and applications and compiles and links information from U.S. Army Corps of Engineers (USACE) manuals. Herein are summaries of foundational USACE documents (e.g., Draft Best Practices in Dam and Levee Safety Risk Analysis, Version 3.0 (Bureau of Reclamation and USACE 2012), U.S. Army Corps of Engineers Emergency Flood Fight Training Manual (USACE 2010), and other USACE publications.

The USACE recommends layers of proven, reliable flood protection. Currently, adequate data and analyses do not exist for USACE recommendation of incorporated or temporary barriers if other proven, exterior structural approaches are possible. The use of incorporated barriers for flood protection at nuclear power facilities is inherently unreliable and inappropriate; however, incorporated barriers may supplement a complete flood protection strategy. Temporary barriers may also supplement a complete flood protection strategy but are not a substitute for exterior barriers.

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Preface

This study was conducted for the Nuclear Regulatory Commission under Project Number V6264, Technical Basis for Flood Protection at Nuclear Power Plants. The technical monitor was Jacob Philip (NRC Project Manager).

The work was performed by the River Engineering Branch (HFR) of the Flood and Storm Protection Division (HF), U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory (ERDC-CHL). At the time of publication, Dr. Loren Wehmeyer was Chief, CEERD-HFR; Dr. Ty V. Wamsley was Chief, CEERD-HF; and William R. Curtis, CEERD-HZT was the Technical Director for Flood and Storm Damage Reduction. The Deputy Director of the ERDC Coastal and Hydraulics Laboratory was Dr. Kevin Barry, and the Director was José E. Sánchez.

At the time of publication, LTC John T. Tucker III was the Acting Commander of ERDC, and Dr. Jeffery P. Holland was the Director.

Executive Summary

Current regulatory guidance on flood protection at nuclear power plants is contained in the Regulatory Guide 1.102, *Flood Protection for Nuclear Power Plants* (Nuclear Regulatory Commission 1976). Regulatory Guide 1.102 requires that structures, systems, and components important to safety be designed to withstand the effects of natural phenomena such as floods, tsunami, and seiches without loss of capability to perform their safety functions. These requirements remain the same in 2013 as they did in 1976; however, the current technology, understanding, and practice have changed. This report describes current (2013) flood-protection feature types and applications for protecting safety-related structures, systems, and components. This document compiles and links information from multiple U.S. Army Corps of Engineers (USACE) manuals in order to provide a resource of scientific and technical background concerning various flood-protection structures and their associated risks and reliability. Several chapters and sections in this report summarize foundational USACE documents. These include the *Draft Best Practices in Dam and Levee Safety Risk Analysis, Version 3.0* (Bureau of Reclamation and USACE 2012), *U.S. Army Corps of Engineers Emergency Flood Fight Training Manual* (USACE 2010), and several USACE circulars, manuals, regulations, pamphlets, and technical letters (variously dated) available online at http://publications.usace.army.mil.

The USACE recommends multiple layers of proven exterior structural barriers and interior pumping stations for reliable flood protection. Currently, adequate data and analyses do not exist in order for the USACE to recommend the use of incorporated or temporary barriers if other proven exterior structural approaches are possible. The use of incorporated barriers for flood protection at nuclear power facilities is inherently unreliable and inappropriate. Incorporated barriers may be able to supplement a complete flood-protection strategy, but the reliability of the protection they provide is insufficient. Temporary barriers may be able to supplement a complete flood-protection strategy, but are not a substitute for adequate exterior barriers, interior drainage systems, and pumping stations.

1 Introduction

Current regulatory guidance on flood protection at nuclear power plants is contained in the Regulatory Guide 1.102, *Flood Protection for Nuclear Power Plants* (Nuclear Regulatory Commission 1976). The U.S. Army Corps of Engineers (USACE), Engineer Research and Development Center (ERDC), in cooperation with the Nuclear Regulatory Commission, Office of Nuclear Regulatory Research (NRC), developed this technical basis document as a foundation of reference information for the NRC to be used by NRC staff to update the existing Regulatory Guide 1.102 based on current understanding and practice related to flood protection at nuclear power plants. This document includes references to many USACE reports that are provided for technical background, not for NRC regulatory purposes.

Regulatory Guide 1.102 requires that structures, systems, and components important to safety be designed to withstand the effects of natural phenomena such as floods, tsunamis, and seiches without loss of capability to perform their safety functions. These requirements remain the same in 2013 as they did in 1976; however, the current understanding and practice has changed. This report describes a technical basis for the most commonly used flood-protection structures and applications for protecting safety-related structures, systems, and components.

Flood protection methods for nuclear power plants fall into one of these five categories:

- dry sites
- exterior (primary) barriers
- incorporated (secondary) barriers
- temporary barriers
- interior drainage/pumping systems to accommodate local intense precipitation.

Chapters 2–6 discuss the five categories of flood-protection methods. Dry sites are located above the design basis flooding level (DBFL). The DBFL is the maximum water elevation attained by the controlling flood, including coincident wind-generated wave effects (Nuclear Regulatory Commission 2011). At a dry site, because a site is above the DBFL, all safety-related

structures, systems, and components are not affected by external flooding but are subject to flooding from local intense precipitation. Exterior barriers are natural or engineered structures exterior to the immediate site. Examples of exterior barriers include earthen embankments, sea walls, floodwalls, revetments, and breakwaters. When properly designed and maintained, exterior barriers can produce a site with the flood risk approaching that of a dry site. Incorporated barriers are engineered structures located at the nuclear power plant site/environment interface. Examples of incorporated structures include floodgates, sealed doors, and pumping stations.

Depending on the location of a barrier, some can be categorized as either exterior or incorporated. In these cases, the barriers are included in the exterior (primary) barrier chapter. The distinction between primary (external) and secondary (incorporated) barriers is important because primary flood-protection failures can be mitigated with a separate method prior to the floodwaters contacting the nuclear power plant site/ environment interface. For example, at a wet site where pumping is required to control interior drainage during an external flood event, sandbag levees could be used to protect critical infrastructure within the site during periods when pumps must be taken off-line temporarily for maintenance. Secondary (incorporated) barriers, by definition, do not have this additional layer of external protection. For nuclear power plants, secondary or temporary barriers, in the absence of primary barriers, are not recommended under any circumstances. Incorporated and temporary barriers may be able to supplement a complete flood-protection strategy but are not a substitute for adequate exterior barriers and interior pumping stations.

Chapter 5 covers temporary flood-fighting measures, summarizing the results of a program testing sandbags, in addition to three commercial measures. Chapter 6 covers flooding from locally intense precipitation, including interior drainage concerns. Chapter 7 summarizes the most recent USACE guidance on flood-fighting methods. Chapter 8 covers other issues, including climate change, resiliency for large storm events, and inspection and evaluation. The final chapter, Chapter 9, provides a summary and recommendations.

2 Dry Site

A dry site is defined as a location where all structures are built above the DBFL (the maximum water elevation attained by the controlling flood, including coincident wind-generated wave effects), and therefore safety-related structures, systems, and components are not affected by external flooding. Exterior (primary) barriers are not applicable based on the definition of a dry site. Barriers intended to manage local intense precipitation become the primary barriers at dry sites. Incorporated barriers remain secondary. The qualitative reliability of a dry site is considered very reliable as long as the DBFL does not increase, causing a dry site to no longer be classified as a dry site.

3 Exterior (Primary) Barriers

This chapter includes an overview, design considerations, and a general reliability discussion for current (2013) flood-protection methods. The categories covered in this chapter include the following:

- seawalls
- bulkheads
- revetments
- breakwaters
- levees
- floodwalls.

The qualitative reliability of external barriers alone depends on the design and maintenance of the barriers. Properly designed and maintained barriers alone are insufficiently reliable for protecting nuclear power facilities. However, coupled with properly designed and maintained internal drainage systems and redundant pumping stations, a flood-protection system including the presence of external barriers should be very reliable.

3.1 Coastal protection

Structures are often needed along shorelines to provide protection from wave action and/or to retain in situ soil or fill. Vertical structures are classified as either seawalls or bulkheads, according to their function, while protective materials laid on slopes are called revetments. For more detailed information on coastal protection, see the following engineer manuals (EM) from which this report is based:

- EM 1110-2-1614 *Design of Coastal Revetments, Seawalls, and Bulkheads*
- Technical Report CERC-93-19 *Engineering Design Guidance for Detached Breakwaters as Shoreline Stabilization Structures* (USACE 1993a)
- EM 1110-2-2502 *Retaining and Flood Walls*
- EM 1110-2-2503 *Design of Sheet Pile Cellular Structures Cofferdams and Retaining Structures*
- EM 1110-2-2504 *Design of Sheet Pile Walls*

- EM 1110-2-2906 *Design of Pile Foundations*
- EM 1110-2-1100 *Coastal Engineering Manual*
- EM 1110-2-1617 *Coastal Groins and near shore Breakwaters*.

3.1.1 Seawalls

Seawalls are defined as structures separating land and water areas, primarily designed to prevent erosion and other damage due to wave action. They are frequently built at the edge of the water, but can be built inland to withstand periods of high water. Seawalls are generally characterized by a massive cross section and a seaward face shaped to dissipate wave energy (EM 1110-2-2502)[1]. Seawalls may be either gravity- or pile-supported structures. Common construction materials are either concrete or stone. Seawalls can have a variety of face shapes (Figure 3-1).

Concrete seawall structures are often pile supported with sheetpile cutoff walls at the toe to prevent undermining. Additional rock toe protection may also be used to prevent scour. The seaward face may be vertical, sloped, stepped, or recurved. Rubble-mound seawalls are designed like breakwaters, using a rock size or concrete armor unit that will be stable against the design wave (Figures 3-2, 3-3). Critical design elements include a secure foundation to minimize settlement and toe protection to prevent undermining. The usual steps needed to develop an adequate seawall design follow (adapted from EM 1110-2-1614).

1. Determine the water level range for the site.
2. Determine the wave heights.
3. Select suitable seawall configurations.
4. Design pile foundations using EM 1110-2-2906 *Design of Pile Foundations*.
5. Select a suitable armor unit type and size (rubble seawalls and toe protection).
6. Determine the potential runup to set the crest elevation.
7. Determine the amount of overtopping expected for low structures.
8. Design underdrainage features if they are required.

[1]Engineer Circulars (EC), Engineer Manuals (EM), Engineer Pamphlets (EP), Engineer Regulations (ER), and Engineer Technical Letters (ETL) are listed in References by order of document publication numbers, not by dates; therefore, citations for these publications will consist of document publication numbers only.

9. Provide for local surface runoff and overtopping and runoff and make any required provisions for other drainage facilities such as culverts and ditches.
10. Consider end conditions to avoid failure due to flanking.
11. Design the toe protection.
12. Design the filter and underlayers.
13. Provide for firm compaction of all fill and backfill materials. This requirement should be included on the plans and in the specifications, and due allowance for compaction must be made in the cost estimate.
14. Develop cost estimate for each alternative.

Figure 3-1. Typical face shapes for concrete gravity seawalls.

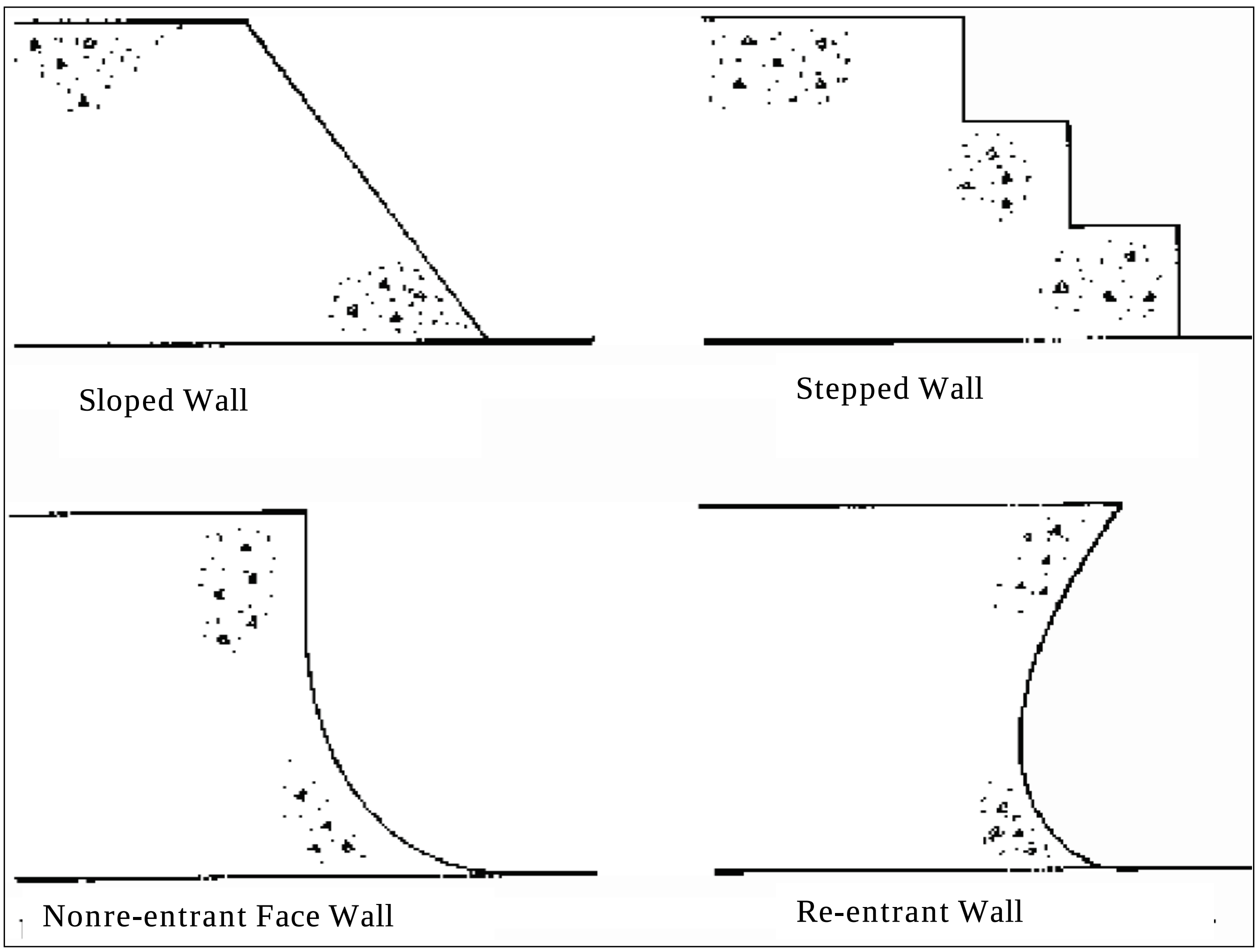


Figure 3-2. Example of a rubblemound seawall on Wallops Island, VA, in 2008.

Figure 3-3. Vertically faced concrete seawall fronted with concrete armor units at Iwakuni, Japan, in 2004.

Seawalls are built parallel to the shoreline as a reinforcement of a part of the coastal profile. Quite often, seawalls are used to protect promenades, roads, and houses placed seaward of the crest edge of the natural beach profile. In these cases, a seawall structure protruding from the natural beach profile must be built. Seawalls range from vertical face structures such as massive gravity concrete walls, tied walls using steel or concrete piling, and stone-filled cribwork to sloping structures with typical surfaces being reinforced concrete slabs, concrete armor units, or stone rubble.

Erosion of upland areas landward of a seawall might be stopped or abated by the structure. However, erosion of the seabed immediately in front of the structure will in most cases be enhanced due to increased wave reflection caused by the seawall. This results in a steeper seabed profile, which subsequently allows larger waves to reach the structure. As a consequence, seawalls are in danger of instability caused by erosion of the seabed at the toe of the structure and by an increase in wave impact, runup, and overtopping. Because of their potential vulnerability to toe scour, seawalls are often used together with some system of toe protection such as groins and beach nourishment. Exceptions include cases of stable rock foreshores and cases where the potential for future erosion is limited and can be accommodated in the design of the seawall.

Seawalls fail for one or more of the following reasons:

1. Design failure occurs when either the structure as a whole, including the foundation, or individual structure components cannot withstand load conditions within the design criteria.
2. Load exceedance failure occurs because anticipated design load conditions were exceeded.
3. Construction failure arises due to incorrect or bad construction or construction materials.
4. Deterioration failure occurs as a result of structure deterioration and inadequate maintenance.

Common failure modes for gravity structures:

1. Toe scour and undermining: because vertically faced structures may increase the reflected wave energy, scour at the toe of the seawall is a common problem that can lead to undermining and instability.

2. Foundation failures: foundation failures include both settling and slip-plain failures.
3. Flanking: although a seawall may halt the erosion of the coastline directly behind the seawall, if the seawall is not properly tied into adjacent hard points, erosion of the coastline will continue at both ends of the seawall leading to flanking and erosion of the land behind the seawall.
4. Erosion of backfill due to overtopping: unless a splash apron or other protective measure is applied, the lands behind the seawall may be eroded by overtopping. Sufficient drainage for the overtopping must also be considered.
5. Spalling or other deterioration of the structure: deterioration is inevitable, and the design must include provision that the deterioration will not affect the functionality of the structure during its design life.

For rubblemound structures, failure modes also include the following:

1. Slope failure due to toe instability or insufficiently sized armor material: toe instability can lead to a slump-type failure of the armor layer, or insufficiently sized armor material will be removed from the matrix by wave action. In either case, the underlayer will be exposed leading to rapid degradation of the structure.
2. Leaching of the substrate through the armor stone: an improperly designed filter layer or tears in a filter fabric will allow the underlying material to leach through the armor layer through wave action. This will result in voids under the armor layer and eventual collapse of the armor.

Maintenance on concrete gravity structures includes sealing any cracks that may develop and repairing any broken sections. Logs and other debris that could be thrown against the structure by wave action should be removed. The toe protection and splash apron, if present, should be inspected and repaired as needed. Seepage drains should be inspected and repaired as needed. Any signs of flanking or backside erosion should be corrected. On rubblemound structures, any holes in the armor layer should be filled, and any evidence of in situ material leaching through the filter layer should be corrected.

3.1.2 Bulkheads

The terms *bulkhead* and *seawall* are often used interchangeably. However, a bulkhead is a retaining wall with the primary purpose of holding or preventing the backfill from sliding while providing protection against light-

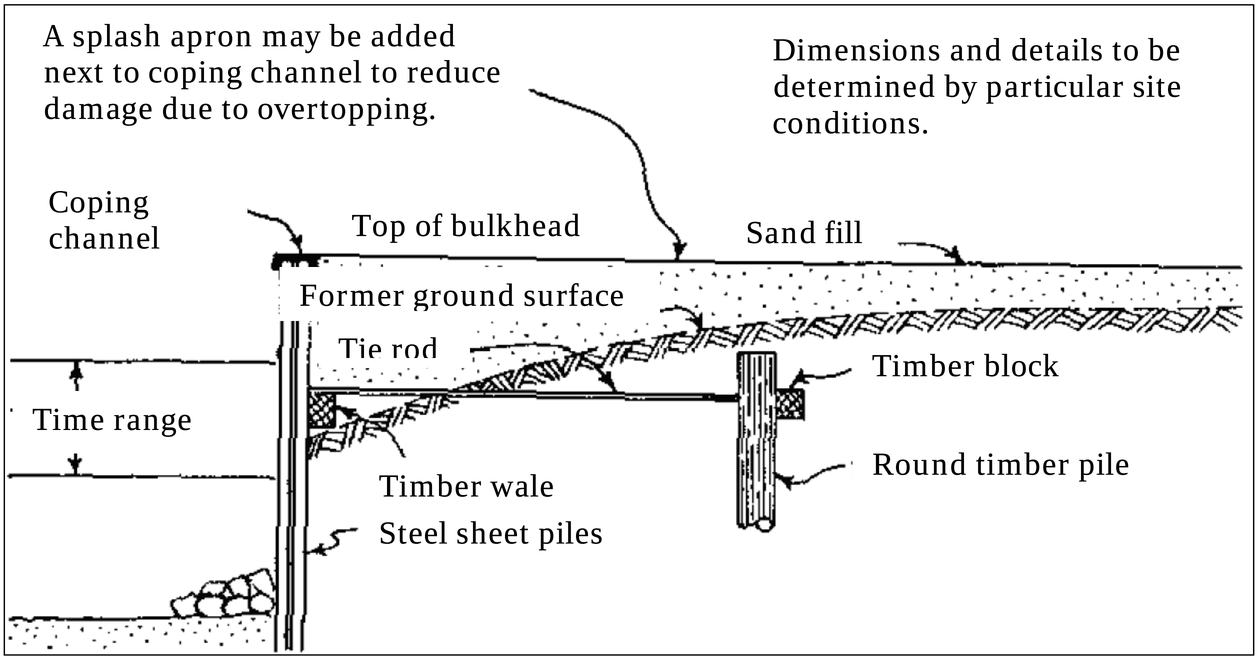
to-moderate wave action (secondary importance). Bulkheads are used to protect eroding bluffs by retaining soil at the toe, thereby increasing stability, or by protecting the toe from erosion and undercutting. They are also used for reclamation projects, where a fill is needed seaward of the existing shoreline, and for marinas and other structures where deep water is needed directly at the shoreline. Bulkhead use is limited to those areas where wave action can be resisted by the bulkhead materials (EM 1110-2-1614).

Bulkheads are typically either cantilevered or anchored sheetpiling or gravity structures such as rock-filled timber cribbing. Cantilevers require adequate embedment for stability and are usually suitable where wall heights are low. Toe scour reduces the effective embedment and can lead to failure. Anchored bulkheads generally are used where greater heights are necessary. Such bulkheads also require adequate embedment for stability but are less susceptible to failure due to toe scour. Gravity structures eliminate the expense of pile driving and can often be used where subsurface conditions hinder pile driving. These structures require strong foundation soils to adequately support their weight, and they normally do not sufficiently penetrate the soil to develop reliable passive resisting forces on the offshore side. Therefore, gravity structures depend primarily on shearing resistance along the base of the structure to support the applied loads. Gravity bulkheads also cannot prevent rotational slides in materials where the failure surface passes beneath the structure. A typical bulkhead section is presented in Figure 3-4. The bulkhead design procedure is similar to that presented for seawalls. In addition, toe protection should be designed using design geotechnical and hydraulic conditions, including wave action and scour potential.

As with seawalls, bulkheads fail for one or more of the following reasons:

1. Design failure occurs when either the structure as a whole, including the foundation, or individual structure components cannot withstand load conditions within the design criteria.
2. Load exceedance failure occurs because anticipated design load conditions were exceeded.
3. Construction failure arises due to incorrect or bad construction or construction materials.
4. Deterioration failure occurs as a result of structure deterioration and inadequate maintenance.

Figure 3-4. Typical steel bulkhead section (EM 1110-2-1614).

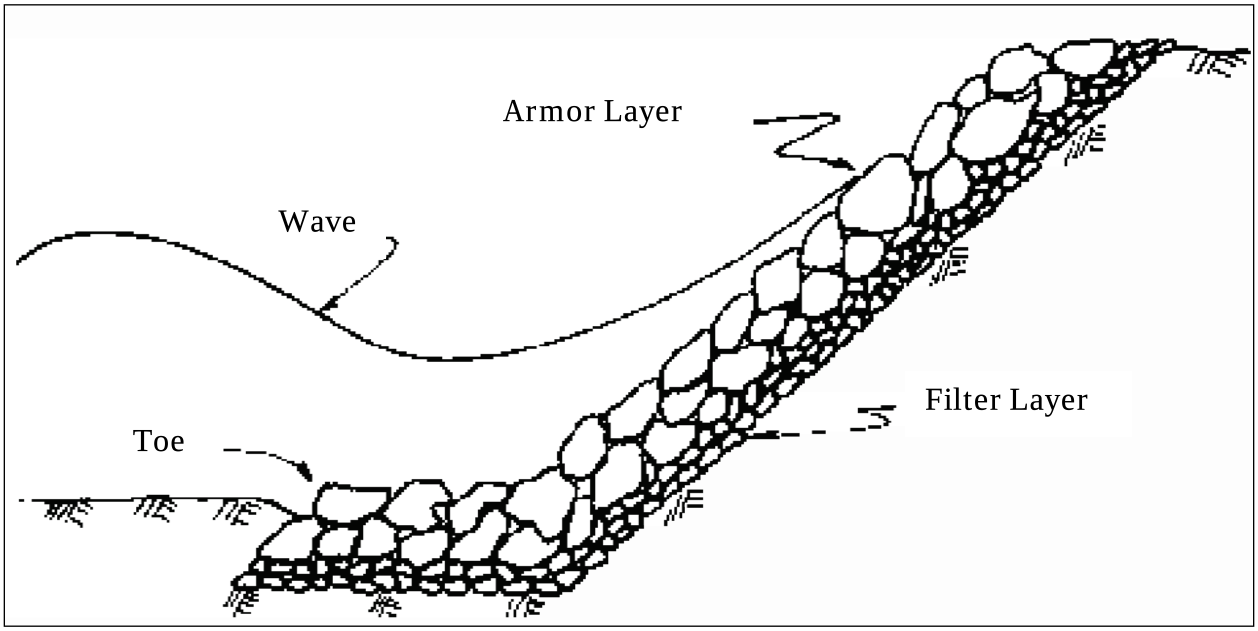


Common failure modes for gravity structures are the same as were listed for seawalls (i.e., toe scour and undermining, foundation failures, flanking, erosion of backfill due to overtopping, and spalling or other deterioration of the structure). Failure modes of tie-back structures also include failure of the tie rod or anchor system. Maintenance on bulkheads is similar to the maintenance of seawalls covered in Section 3.1.1, plus any maintenance that may be required of tie-back or anchoring systems.

3.1.3 Revetments

A revetment is a facing of erosion-resistant material, such as stone or concrete that is built to protect a scarp, embankment, or other shoreline feature against erosion. The major components of a revetment are the armor layer, filter, and toe (Figure 3-5). The armor layer provides the basic protection against wave action, while the filter layer supports the armor, provides for the passage of water through the structure, and prevents the underlying soil from being washed through the armor. Toe protection prevents displacement of the seaward edge of the revetment and prevents scour (EM 1110-2-1614).

Figure 3-5. Typical revetment section (EM 1110-2-1614).



Coastal revetments are onshore structures with the principal function of protecting the shoreline from erosion. Revetment structures typically consist of a cladding of stone, concrete, or asphalt to armor sloping natural shoreline profiles. In the USACE, the functional distinction is made between seawalls and revetments for the purpose of assigning project benefits; however, in the technical literature there is often no distinction between seawalls and revetments.

For revetments in tidal inlets or rivers, the stream velocities may be the factor to determine the revetment size and type. (For additional discussion on revetments used for river and streambank protection, see section 6.3.1 Streambank protection). In most cases, the steepest recommended slope is one unit vertical for each two units horizontal. Fill material should be added where needed to achieve a uniform slope, but it should be free of large stones and debris and should be firmly compacted before revetment construction proceeds. Allowance should be made for conditions other than waves such as floating ice, logs, and other debris. Current velocities may also be important in some areas such as within tidal inlets where wave heights are low. Properly sized filter layers should be provided to prevent the loss of slope material through voids in the revetment stone. If using filter cloth, an intermediate layer of smaller stone below the armor layer may be needed to distribute the load and prevent rupture of the cloth. Economic evaluation of rock revetments should include consideration of trade-offs that result between flatter slopes and smaller stone weights and

the increased costs for excavation that usually result for flatter slopes. Planning and design procedure considerations for coastal projects are described in EM 1110-2-1100, *Part V* and EM 1110-2-1100, *Part VI*.

Revetment armoring may range from concrete slabs-on-grade (rigid) to riprap and quarrystone (flexible). Rigid armors tend to be more massive but are generally unable to accommodate settlement or adjustments of the underlying materials. Flexible armor is constructed with lighter individual units that can tolerate varying amounts of displacement and shifting.

The usual steps needed to design an adequate revetment follow (adapted from EM 1110-2-1614):

1. Determine the water level range for the site.
2. Determine the wave heights.
3. Select suitable armor alternatives to resist the design wave.
4. Select armor unit size.
5. Determine potential runup to set the crest elevation.
6. Determine amount of overtopping expected for low structures.
7. Design underdrainage features if they are required.
8. Provide for local surface runoff and overtopping runoff and make any required provisions for other drainage facilities such as culverts and ditches.
9. Consider end conditions to avoid failure due to flanking.
10. Design toe protection.
11. Design filter and underlayers.
12. Provide for firm compaction of all fill and backfill materials. This requirement should be included on the plans and in the specifications. Also, due allowance for compaction must be made in the cost estimate.
13. Develop cost estimate for each alternative.

Revetments are typically constructed as sloping-front, flexible rubble-mound structures that are able to adjust to some toe and crest erosion. In the U.S., pattern-placed block slopes are commonly found on revetments. The stability of the slope is dependent on an intact toe support. In other words, loss of toe support will likely result in significant armor layer damage, if not complete failure of the armored slope.

Common failure modes for rubble-mound structures include the following:

- slope failure due to toe instability or insufficiently sized armor material
- leaching of the substrate through the armor stone due to improperly designed filter layer or tears in filter cloth
- toe scour
- flanking
- erosion landward of structure due to overtopping.

Maintenance of rubble-mound structures includes filling any holes in the armor layer and ensuring that the filter layer is preventing leaching of the in situ material. The toe protection should be inspected and repaired as needed, and any problems with flanking or erosion landward of the structure from overtopping should be corrected.

3.1.4 Breakwaters

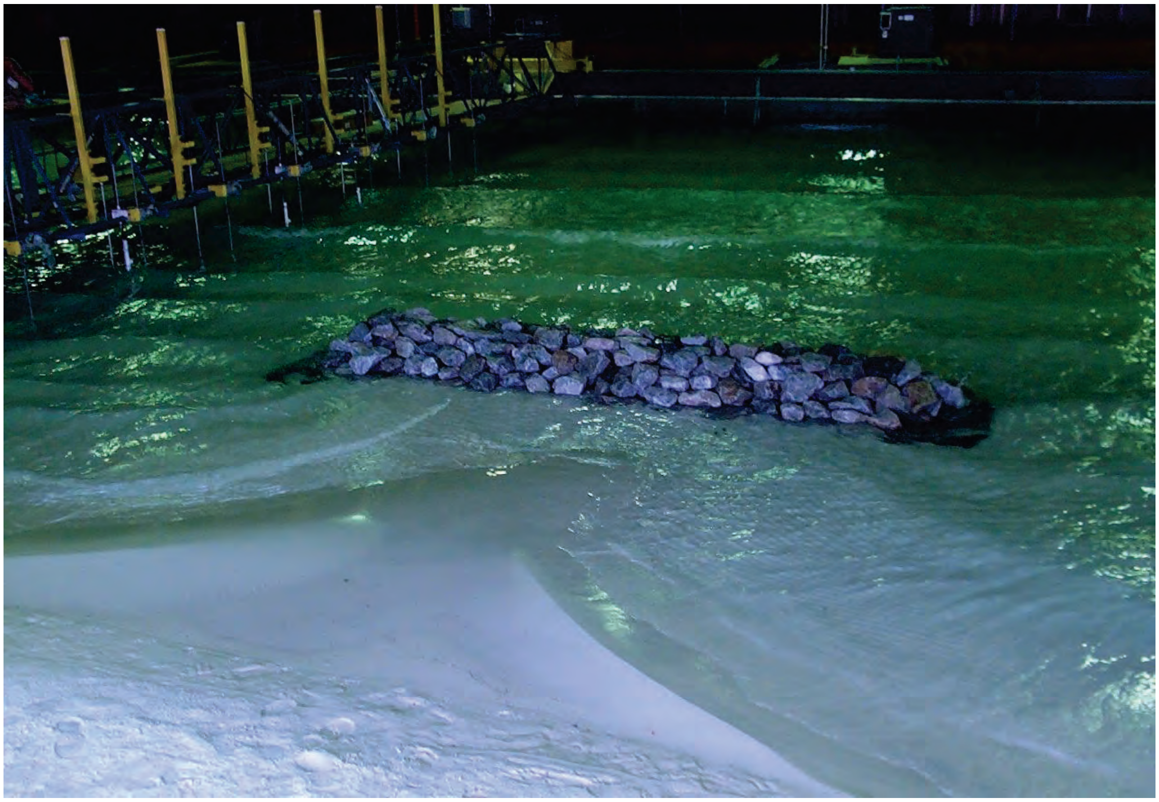
Breakwaters differ from seawalls, revetments, and bulkheads in that a breakwater will have water on both sides of the structure. There are numerous variations of the breakwater concept (EM 1110-2-1617). Breakwaters may be shore attached, detached, submerged, or emergent. Shore-attached breakwaters are commonly used to form a protective barrier around mooring areas in ports and harbors. The breakwater forms a physical barrier to wave action providing an area of reduced wave action for mooring (see Figure 3-6). Detached breakwaters are constructed at a significant distance offshore and are typically used for shoreline protection by reducing the amount of wave energy reaching the shoreline. Reef breakwaters are a type of detached breakwater designed with a low crest elevation and homogeneous stone size, as opposed to the traditional multilayer cross section. Low-crested breakwaters can be more suitable for shoreline stabilization projects due to increased tolerance of wave transmission and reduced quantities of material necessary for construction. Other types of breakwaters include headland breakwaters or artificial headlands, which are constructed at or very near to the original shoreline. A headland breakwater is designed to promote beach growth out to the structure, forming a tombolo or periodic tombolo, and tends to function as a transmissible groin. Another type of shore-parallel offshore structure is a submerged sill or perched beach. A submerged or semisubmerged sill reduces the rate of offshore sand movement from a stretch of beach by acting as a barrier to shore-normal transport. The effect of submerged sills on waves is relatively small due to their low crest elevation.

Figure 3-6. Shore attached breakwaters protecting a small boat dock in Corozal, Panama, in 2005.

Detached breakwaters are generally shore-parallel structures that reduce the amount of wave energy reaching the protected area by dissipating, reflecting, or diffracting incoming waves. The structures dissipate wave energy similar to a natural offshore bar, reef, or nearshore island. The reduction of wave action promotes sediment deposition shoreward of the structure. Littoral material is deposited and sediment retained in the sheltered area behind the breakwater. The sediment will typically appear as a bulge in the beach planform termed a salient, or a tombolo if the resulting shoreline extends out to the structure (Figure 3-7).

Reef breakwaters are shore-parallel, submerged structures built with the objective of reducing the wave action on the beach by forcing wave breaking over the reef and dissipation of wave energy through turbulence within the reef. Reef breakwaters are normally rubble-mound structures constructed as a homogeneous pile of stone or concrete armor units. The breakwater can be designed to be stable, or it may be allowed to reshape under wave action. Reef breakwaters might be narrow crested like detached breakwaters in shallow water or, in deeper water, wide crested with lower crest elevation. Besides triggering wave breaking and subsequent energy dissipation, reef breakwaters can be used to regulate wave action by refraction and diffraction. Reef breakwaters represent a nonvisible hazard to swimmers and boats.

Figure 3-7. Reef breakwater and the corresponding salient developed in the USACE Large-Scale Laboratory Facility for Sediment Transport Research.



When used for shore protection, breakwaters are built in nearshore waters and usually oriented parallel to the shore. The layout of breakwaters is determined by the size and shape of the area to be protected as well as by the prevailing directions of storm waves, net direction of currents and littoral drift, and requirements for maneuverability of navigation vessels.

The cost of building a structure with sloping sides increases dramatically with increasing water depth. Cost of building a structure with rock or concrete armor units will rise with increasing wave climate due to the larger size of units required. For these reasons, breakwaters in deep water are frequently constructed of concrete with vertical sides, either with sand-filled concrete caissons or stacked, massive concrete blocks. The concrete caissons are often built on a high mound of quarry rock for economical reasons. These breakwaters are called *composite structures*. The upper part of the concrete structure might be constructed with a sloping front to reduce the wave forces. For the same reason, the front wall might be perforated with a wave chamber behind to dissipate wave energy. Smaller vertical structures might be constructed of steel sheetpiling backfilled with soil or built as a rock-filled timber cribwork or wire cages. In milder wave

climates, sloping reinforced concrete slabs supported by batter piles are also applied.

Rubble-mound breakwaters are subject to the same failure modes as revetments with additional failure modes for the lee side due to overtopping or transmission through the structure. Vertically faced breakwaters have the same failure modes as seawalls with the additional failure modes of overturning or sliding. Rubble-mound breakwaters have similar maintenance requirements as rubble-mound revetments discussed in Section 3.1.4. The concrete portions of composite breakwaters will have maintenance requirements similar to concrete seawalls discussed in Section 3.1.1.

3.1.5 Coastal design and reliability

All projects accept some level of failure probability associated with exceedance of design load conditions, but failure probability increases at project sites where little prototype data exist upon which to base the design. These cases may require a conservative factor of safety (for information on probabilistic design, see EM 1110-2-1100, *Part V-1-3* and EM 1110-2-1100, *Part VI-6*). Conventional design practice for coastal structures is deterministic in nature and is based on the concept of a design load that should not exceed the resistance (carrying capacity) of the structure. Resistance is defined using a preselected probability of failure that considers variability in the quality of construction and the consistency of the construction materials. In most cases, the resistance is defined in terms of the load that causes a certain design impact or damage to the structure, and it is not given as an ultimate force or deformation. This is because most of the available design formulae only give the relation between wave characteristics and some structural response, such as runup, overtopping, armor layer damage, etc.

Almost all coastal structure design formulae are semiempirical and based mainly on central fitting to model test results. The scatter in test results is not considered in general because the formulae generally express only the mean values. Consequently, the applied characteristic value of the resistance is then the mean value and not a lower fraction as is usually the case in other civil engineering fields. The only contribution to a safety margin in the design is inherent in the choice of the return period for the design load. The exception is when the design curve is fitted to the conservative side of the data envelope to give a built-in safety margin.

In addition to design load probability, a safety factor might be applied as well, in which case the method is classified as a Level I (deterministic/quasi-probabilistic) method. However, this approach does not allow determination of the reliability (or the failure probability) of the design; consequently, it is not possible to optimize structure design or avoid overdesign of a structure. In order to overcome this problem, more advanced probabilistic methods must be applied where the uncertainties (the stochastic properties) of the involved loading and strength variables are considered.

Level II methods generally transform correlated and non-normally distributed variables into uncorrelated and standard normal distributed variables, and reliability indices are used as measures of the structural reliability. Methods where the actual distribution functions for the variables are taken into account are denoted as Level III methods. (For more information about Level I, Level II, and Level III methods, as well as planning and design procedure considerations for coastal projects, refer to EM 1110-2-1100, *Part V* and EM 1110-2-1100, *Part VI***.**)

3.2 Riverine and other noncoastal protection

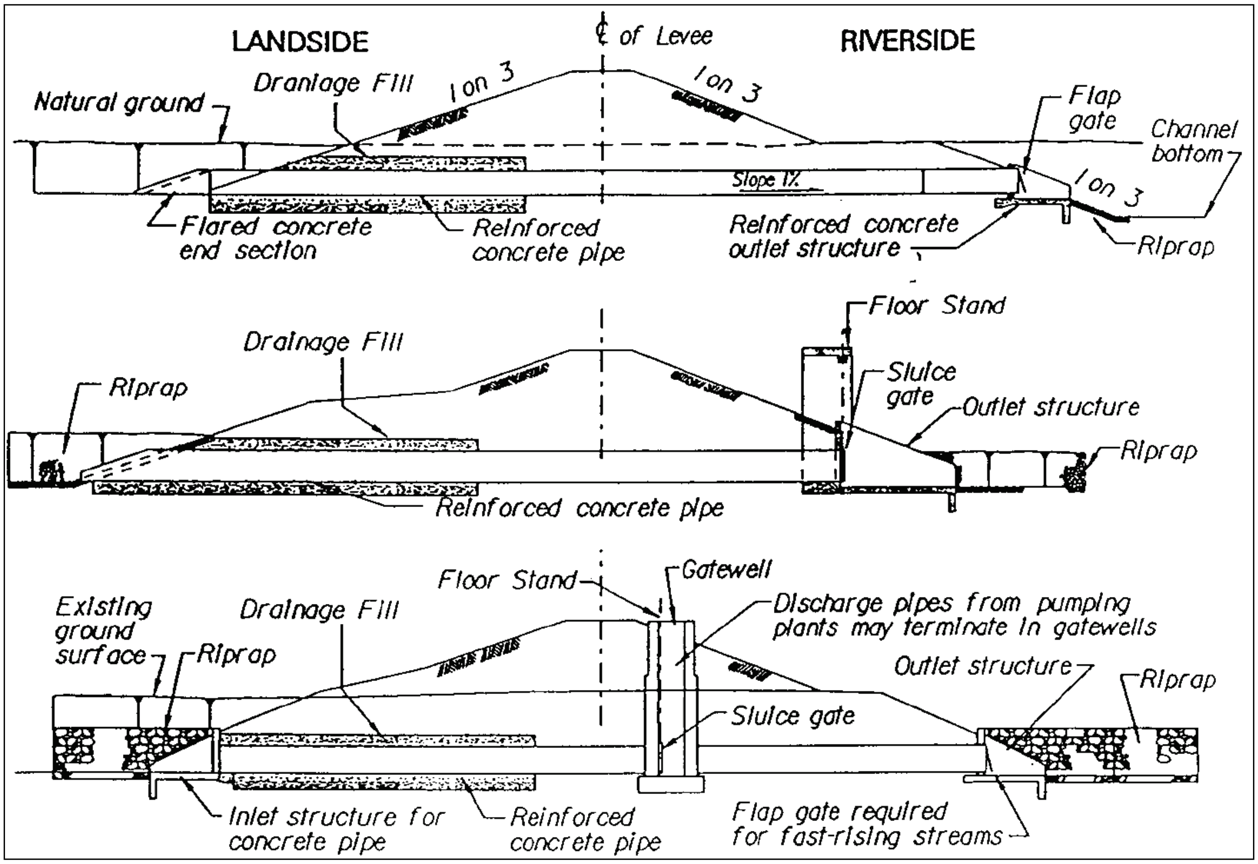
Structures are often needed along rivers, streams, and other interior water bodies (lakes, bays, etc.) to provide protection from flooding. Riverine structures are generally earthen embankments or flood walls. (For more detailed information on riverine and other noncoastal protection, see the following engineer manuals from which this report section is based: EM 1110-2-1913, EM 1110-2-2502, EM 1110-2-2503, and EM 1110-2-2504.)

3.2.1 Earthen embankments

Earthen embankments come in a variety of configurations that vary in design and construction details. A levee is defined as an embankment whose primary purpose is to furnish flood protection from seasonal high water and which is therefore subject to water loading for periods of only a few days or weeks a year. For some large rivers, the period of flooding may exceed one month. Embankments that are subject to water loading for prolonged periods (longer than normal flood-protection requirements) or permanently should be designed in accordance with earth dam criteria rather than the levee criteria given herein" (EM 1110-2-1913). Illustrative sections of levees with engineered conduit penetrations (e.g., drainage structures) are shown in Figure 3-8. EM 1110-2-1913 provides a

comprehensive reference concerning the geotechnical design and construction of levees. The hydrologic and hydraulic design can be performed using references such as *Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America* (Nuclear Regulatory Commission 2011) and EM 1110-2-1416.

Figure 3-8. Levee sections with conduit penetrations (EM 1110-2-1913).



Numerous local factors must be considered in levee design, and no specific step-by-step procedure covering details of a particular project can be established. However, general, logical steps based on successful USACE past projects (adapted from EM 1110-2-1913) are listed below. These steps cover the geotechnical design and use the top of levee profile determined by the hydrologic and hydraulic analyses.

1. Determine the water level range and wave heights for the site.
2. Conduct geological study based on a thorough review of available data including analysis of aerial photographs. Initiate preliminary subsurface explorations.

3. Analyze preliminary exploration data, and from this analysis, establish preliminary soil profiles, borrow locations, and embankment sections.
4. Initiate final exploration to provide additional information on soil profiles, undisturbed strengths of foundation materials, and more detailed information on borrow areas and other required excavations.
5. Using the information obtained in step 3, determine both embankment and foundation soil parameters and refine preliminary sections where needed, noting all possible problem areas, and compute rough quantities of suitable material and refine borrow area locations.
6. Divide the entire levee into reaches of similar foundation conditions, embankment height, and fill material and assign a typical trial section to each reach.
7. Analyze each trial section as needed for underseepage and through seepage, slope stability, settlement, and trafficability of the levee surface.
8. Design special treatment to preclude any problems as determined from step 6. Ascertain surfacing requirements for the levee based on its expected future use.
9. Based on the results of step 7, establish final sections for each reach.
10. Compute final quantities needed; determine final borrow area locations.
11. Design embankment slope protection.

The principal causes of embankment failure are overtopping and excessive seepage. Embankments should be designed to overtop at locations where the overtopping does not affect critical infrastructure (usually the downstream end). Most embankment failures are caused by excessive seepage, internal erosion, or slope instability. Such failures tend to occur rapidly and with little or no warning—leaving little opportunity for evacuation prior to flooding. Failures caused by overtopping are often foreseeable and tend to progress more slowly, and in some cases can be prevented through aggressive flood fighting. Failures from overtopping provide much better opportunity to successfully evacuate the threatened area and to take steps to minimize damage. (For more information on earthen embankment seepage principles and analysis, refer to EM 1110-2-1901 and ETL 1110-2-569.)

Seepage is defined as the movement of water through the interstitial soil matrix located anywhere within an embankment, its foundation, or its abutments. Seepage is differentiated from leakage, which is the unintentional flow of water through holes or cracks. As an example, a broken pipe (conduit) will leak, and the resulting flow of water into the

surrounding soil will develop an interstitial flow path (i.e., seepage) of which the direction and quantity is directly proportional to the soil's hydraulic conductivity (permeability) and pore water pressure. Seepage discharge may vary in appearance from a wet surface area having slightly denser vegetation growth to one having measurable water flow rates and substantial water volume. In some cases, seepage may be harmless, but in others, it may be extremely serious and immediate remedial action must be taken to prevent a seepage-erosion-induced breaching failure. Seepage must be considered in the design of interior drainage/pumping systems. This is discussed in section 6.1 Interior Drainage Systems.

Water seepage only threatens the structural integrity of an embankment when the seepage begins to transport soil particles within the embankment or its subsurface proximity. When the soil particle movements are not prevented (either by design or by random chance), the subsequent sequence of events may eventually lead to unsatisfactory performance (i.e., breaching failure). The time-to-failure due to such uncontrolled soil particle movements can be rapid, or it can be prolonged. Fell et al. (2003) estimated the elapsed time between first observing an internal soil displacement anomaly (usually evidenced by a muddy flow or an increase in seepage) to eventual failure occurred over time spans ranging from fewer than 3 hours to years. Historically, most seepage-induced failures happen rapidly (Charles 1997). These facts have substantial implications for properly conducting seepage detection, collection, measurement, monitoring, and evaluation aspects for any flood-protection embankment.

If the embankment was constructed with an internal filter or drain system, any potential seepage water discharge should appear in the embankment downstream toe area or toe drain. Toe drains, chimney drains, and blanket drains are designed to intercept the through-seepage discharge and collect and convey it for measurement. Properly designed filters restrain particles from moving with the seepage flow. If the embankment does not have a filter or drain system, seepage water may appear anywhere on the downstream face. In the absence of a properly designed and constructed filter or drain system, the seepage has the potential to erode embankment or foundation materials through internal erosion or piping. Seepage through embankments must always be monitored, measured, and evaluated to verify performance of the core, filter, and drain systems.

Internal erosion can occur as water flows through the internal fractures, cracks, and voids of the earthen embankment. If the water physically removes soil material from within the embankment or foundation, there is potential for an erosion-induced embankment failure. Internal erosion is the primary mechanism responsible for seepage erosion-induced damage. Seepage incidents are reported more often than internal erosion failure incidents because for seepage incidents, the erosion progression is internally terminated before a breach mechanism can develop (Federal Emergency Management Agency (FEMA) 2000).

When an earthen embankment is encountered by floodwaters, storm surge, or wave action, the protected side (inner slope) may erode, and the progressive soil loss may eventually cause a breaching failure. This overtopping erosion process has been empirically observed during natural disasters such as floods and hurricanes. There is also the risk that the embankment wave-side (outer) slope will erode and cause breaching due to wave action, based on historical observations. Whether resisting outer-slope wave attack or inner-slope overtopping forces (or combinations thereof), the embankment structure resilience depends in part on the likelihoods of erosion initiation, progression, and subsequent breaching failure.

The most common slope stability failure mechanisms include shear failure, surface sloughing, excessive deformation, and seismically induced liquefaction. A shear failure involves sliding of a portion of an embankment, or an embankment and its foundation, relative to the adjacent mass. A shear failure is conventionally considered to occur along a discrete surface and is so assumed in stability analyses, although the shear movements may in fact occur across a zone of appreciable thickness. Failure surfaces are frequently approximately circular in shape. Where zoned embankments or thin foundation layers overlying bedrock are involved, or where weak strata exist within a deposit, the failure surface may consist of interconnected arcs and planes. Surface sloughing is considered a maintenance problem because it usually does not affect the structural capability of the embankment. However, repair of surficial failures can entail considerable cost. If such failures are not repaired, they can become progressively larger and may then represent a threat to embankment safety.

To avoid excessive deformations, particular attention should be given to the stress-strain response of cohesive embankment and foundation soils during design. When strains larger than 15% are required to mobilize peak

strengths, deformations in the embankment or foundation may be excessive. If cohesive soils are compacted too dry, and they later become wetter while under load, excessive settlement may occur. Also, compaction of cohesive soils dry of optimum water content may result in brittle stress-strain behavior and cracking of the embankment. Cracks can have adverse effects on stability and seepage. When large strains are required to develop shear strengths, surface movement measurement points and piezometers should be installed to monitor movements and pore water pressures during construction, in case it becomes necessary to modify the cross section or the rate of fill placement. Engineer Technical Letter (ETL) 1110-2-556 describes techniques for probabilistic analyses and their application to slope stability studies.

Seismically induced soil liquefaction, or a significant reduction in soil strength and stiffness as a result of shear-induced increase in pore water pressure, is an earthquake damage concern for earthen embankments. Most instances of liquefaction have been associated with saturated loose sandy or silty soils. Loose, gravelly soil deposits also are vulnerable to liquefaction. Cohesive soils with more than 20% of particles finer than 0.005 millimeter (mm), liquid limit of 34 or greater, or with the plasticity index of 14 or greater are generally considered not susceptible to liquefaction. Evaluation and mitigation for seismic performance of earthen embankment systems have generally had low priority in the past, except for earthen embankments with a high likelihood of having coincident high water and earthquake loading, such as many earthen embankments in the California Delta. The current approach for earthen embankments with infrequent high water is for seismic performance evaluation to occur at typical water surface elevations. Flood risk coincident with seismic performance has typically been addressed with emergency response, interim and long-term repairs following the earthquake, and/or seismic remediation prior to the earthquake.

Several other types of slope movements, including rock falls, topples, lateral spreading, flows, and combinations of these, are not controlled by shear strength. Earthen embankments are generally not designed for these types of mass movements, but the possibility of their occurrence should not be ignored. Earthen embankment design guidance is provided in EM 1110-2-1913, ETL 1110-2-569, ETL 1110-2-570, and the *Geotechnical Levee Practice Standard Operating Procedure* for USACE District offices. Successful design requires consistency in the design process. Appropriate

safety factors are inseparable from the procedures used to measure shear strengths, analyze stability, and evaluate seepage.

A deterministic design approach based on the expected water surface elevation for a given flood frequency event is typically used to certify earthen embankments for accreditation by FEMA. The earthen embankment must be analyzed for erosion, stability, seepage, and settlement based on this water surface and a minimum amount of freeboard (typically 3 feet (ft)) provided above this water surface elevation. As little as 2 ft of freeboard may be allowed if the uncertainty in flow and stage is characterized and justifies less than 3 ft. of freeboard. In recent years, the USACE has been developing and transitioning toward a semiprobabilistic approach. The semiprobabilistic approach is a risk-based geotechnical analysis method to replace the deterministic geotechnical analysis method contained in guidance documents, including EM 1110-2-1913, ETL 1110-2-569, and ETL 11102-570.

The presence of nongrass vegetation such as trees and shrubs may inhibit satisfactory performance of an earthen embankment's functions. Tree roots, for example, penetrating into a levee structure, may increase the probability of levee underseepage. ETL 1110-2-571 provides guidelines for vegetation management at embankments and floodwalls. A minimum nongrass vegetation-free zone beyond the levee toe (or distance to edge of normal water surface) of 15 ft is codified in that document.

Maintenance issues on earthen levees include maintaining the grass covering while inhibiting and removing nongrass vegetation. Drainage channels or structures need to be inspected and repaired as necessary. The levee must be regularly inspected for signs of seepage and corrective measures taken as needed. Damage from overtopping or weather should be repaired. Animal burrows need to be filled and burrowing animals relocated.

3.2.2 Flood walls

There are numerous floodwalls in place across the Nation's system of levees. In general, floodwalls are used when there is insufficient land to place an earthen levee up to the required level of protection. They are more prevalent in urban areas where real estate is at a premium, but they may have limited use in some rural areas as well. There are a wide variety of floodwalls, but the overwhelming majority of these are I-walls and T-walls. Other less

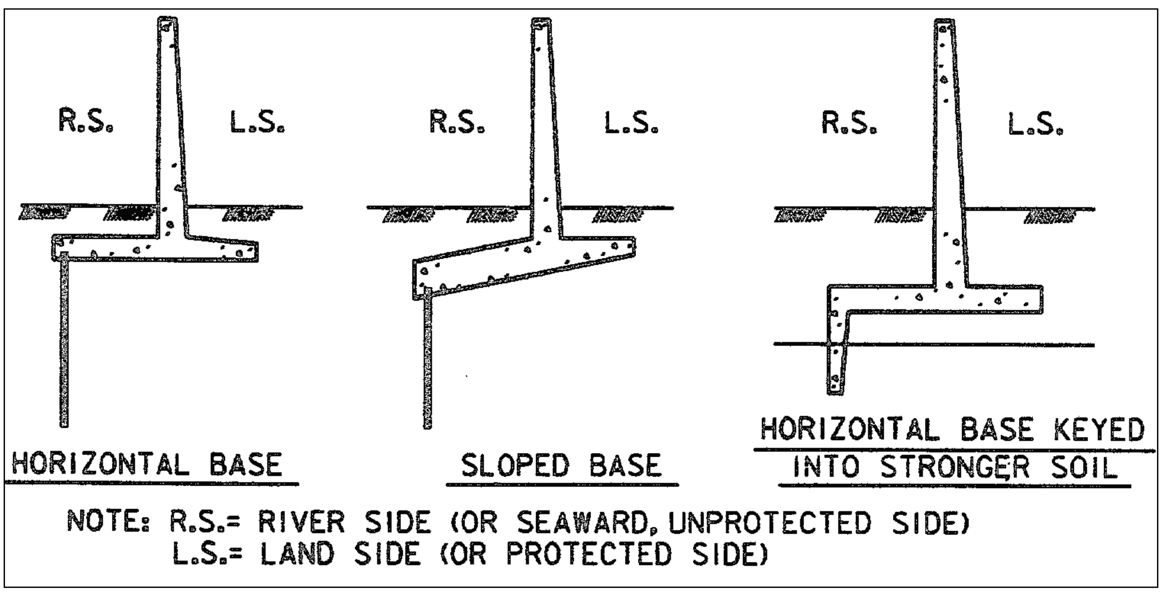
common floodwall types include L-walls, buttress/counterfort walls, and gravity- style walls. L-walls can be assessed with similar methods as those outlined in this T-wall section of this report. Gravity walls can be assessed for stability using the general wedge methodology as EM 1110-2-2100. Buttress (counterfort) walls are essentially T-walls with a structural member on intervals to help support the stem of the wall. These are more difficult to analyze than traditional T-walls because they have different failure mechanisms such as moment and shear failure of the buttress (counterfort) section.

T-walls are one of the predominant types of floodwall in use. As noted earlier, T-walls get their name from the fact the cross-sectional area takes the general shape of an inverted “T”. T-walls are generally used in lieu of I-walls when the heights required for flood protection become larger than an I-wall can safely handle. Only a review of the as-built plans will allow one to determine whether a wall is a T-wall or an I-wall. Confirmation is not possible by simply looking at it from the ground. When the foundation conditions are undesirable, T-walls are often pile founded for stability purposes. The piles transfer the load to better soil/rock conditions founded below the unsuitable foundation soils near the surface. In addition, many T-walls have sheetpile cutoff walls located on the riverward (heel side) to improve underseepage performance. Some T-walls may have sloped base slabs to improve global stability. Relief wells and/or toe drains on the protected side may also be present to help control underseepage. Examples of three different T-wall cross-sections, taken from EM 1110-2-2502, are shown in Figure 3-9 for reference. The external loads acting on most flood-protection T-walls are usually relegated to earth and water pressures. The weight of the concrete is also considered in the global stability analysis.

Several different water levels will likely have to be evaluated in order to develop a system-response curve (probability of wall failure vs. water level) for a risk analysis. There are several important considerations for determining the water levels to evaluate as part of the risk analysis. An estimate of the water surface profile compared to an accurate top of levee/floodwall along the line of protection will help determine how high the water likely will rise against the floodwall section being evaluated before incipient overtopping possibly occurs at another location along the line of protection. A few other key points regarding selecting water elevations for risk analysis purposes include the following:

1. Ensure the datum being used for water surface elevation estimate is consistent with the top of levee/floodwall profile. Different datums have been used throughout the U.S.
2. Ranges of loading will most likely be required for the risk analysis. A good starting point is 25%, 50%, 75%, and 90% of the wall overtopping.
3. Evaluate for the midpoint of the range since the failure probability that is developed is used to represent the entire range. The frequency of this loading also needs to be taken into account from a risk analysis perspective. It is important to have tighter ranges at water elevations that are likely to be critical from a performance standpoint. For example, to assess the performance for the 50%–75% exposed height range, the assessment should be for the 62.5% exposed height and use those results for the entire range. This will be done for each range evaluated.
4. The water levels used to develop failure probabilities for the wall section need to be consistent with the levels used for the consequence estimates. A relation between consequences and water elevation should be developed. The analysis for both the wall performance and consequences needs to cover the entire range of water elevations considered for the risk analysis.

Figure 3-9. Inverted T-type cantilever flood walls (EM 1110-2-2502).



There are several failure modes that are considered viable for levee T-walls, but they can generally be separated into three broad categories: global instability, structural performance, and underseepage/piping. Global instability refers to overturning, sliding, and bearing capacity. Global instability failures can occur before or after overtopping of a floodwall. If the floodwall holds and then overtops, passive resistance on

the protected side can be eroded away, leading to global instability. Structural performance relates to excessive moment and shear forces failing the structural wall section. Underseepage and piping involves the movement of foundation soils below the wall causing a loss of wall foundation support and subsequent stability failure.

The presence of trees and significant vegetative growth immediately adjacent to floodwalls has the potential to adversely affect stability of floodwalls in a variety of ways. This could be vegetation on either side of the wall. A *safe distance* needs to be provided from the foundation of the wall to any significant vegetation; unfortunately, there is no *preset* safe distance that will account for all situations, and each must be judged in the context of how a tree might adversely affect floodwall stability in its given environment. The 15 ft vegetation-free zone within USACE guidance is specific to maintenance and inspection requirements. This distance is not necessarily indicative of how vegetation may affect floodwall stability and should not be taken as such. There are instances where certain types of vegetation within 15 ft may not be harmful to the performance of the floodwall, just as there are instances where vegetation greater than 15 ft away from the floodwall's foundation could potentially fail the wall. Careful engineering judgment is required to evaluate each situation on its own merits. When large trees and/or trees with significant root systems are located in the vicinity of floodwalls, careful consideration must be made of how they might adversely affect performance. A few situations to consider include the following:

1. Trees with large root systems extending below floodwalls have the potential to *jack* or lift the wall, potentially causing a wide range of failure issues such as cracking, separation of joints, or wall failure.
2. Large trees adjacent to walls can topple over and structurally damage a wall particularly when surrounding soils are already saturated from heavy rains and flooding.
3. Floodwalls with toe drainage systems in place to relieve uplift pressures for wall stability can be damaged either by tree roots penetrating the toe drain system or by having an uprooted tree dislodge the drainage system, rendering it ineffective.
4. Floodwalls requiring passive resistance for stability can also fail if a large soil mass on the protected side is removed by an overturned tree.

Inland flood walls typically are installed along a riverbank and are subjected to design loadings (pool to freeboard line) for periods of hours or days (long-term loadings). Coastal flood walls are primarily subjected to short-term loadings (waves from hurricanes along with wind/tide high water surges, tsunamis, etc.). The wave loadings are dynamic in nature and act upon the structure for only a few seconds each. Concurrent high winds can prevent any emergency maintenance during a storm. Utility line crossings through a flood wall require careful attention to allow for independent movement of the utility lines and the wall, which requires special expansion joint details.

Water-retaining structures are subject to through-seepage, underseepage, and seepage around their sides or ends. Seepage control is a primary consideration of flood wall design. Uncontrolled seepage may result in water pressures and uplift forces on the wall base in excess of design assumptions and consequent structural instability. Excessive porewater pressures in foundation materials near the landside toe of a wall may create *quick* conditions evidenced by sand boils or heaving. Emerging seepage may have sufficient velocity to move cohesionless foundation materials and erode the wall foundation (piping). Seepage control entails the design of measures to ensure that seepage pressures and velocities are maintained below tolerable values. Properly controlled seepage, even if quantities are large, can present no hazard. Flood walls in congested areas often require seepage to be pumped out of the protected area. While the seepage quantity is often small compared to other sources, it is occasionally appropriate to consider seepage control measures for the purpose of reducing seepage quantities.

Inadequate seepage control may jeopardize the stability of a flood wall. In flood walls, control of through-seepage is provided for by water stops. Seepage around the wall is controlled by specially designed and constructed levee wrap-around sections. Flood walls are usually provided with a toe drain to control local underseepage along the flood wall base. As flood walls are usually founded on alluvial materials, pervious zones of significant thickness are often present at some depth below relatively impervious top stratum materials and may be hydraulically connected to the river. Because of the horizontal stratification of alluvial deposits, the horizontal permeability may be greatly in excess of the vertical permeability.

The combination of these conditions may allow seepage to be readily conducted landward beneath the flood wall. Where flood walls are underlain by such pervious strata (the usual case), analysis may indicate the need for underseepage controls in addition to the toe drain. Underseepage control measures vary because the selection and design of an appropriate control scheme is highly dependent on site-specific conditions, particularly the stratification and permeability of foundation materials, availability of right-of-way, and local construction practices and costs.

Careful attention must be given to wall monoliths that have loading, support, or other conditions that vary along the length of the monolith. These monoliths, which may include closure structures, pipeline crossings, corner structures, etc., must be analyzed as complete three-dimensional entities instead of the usual two-dimensional unit slices. Planning and design procedure considerations for floodwall projects are described in EM 1110-2-2502 and EM 1110-2-2102.

EM 1110-2-6053 covers requirements for the seismic design and evaluation of plain and reinforced concrete hydraulic structures. The types of concrete hydraulic structures addressed in this manual include dams, U- and W-frame locks, gravity walls, and intake/outlet towers. The guidelines are also applicable to spillways, outlet works, hydroelectric power plants, and pumping plants. The structures may be founded on rock, soil, or pile foundations and may or may not have back-fill soil.

Potential failure modes of cantilever sheet piling walls are discussed in EM 1110-2-2504. An I-wall is a slender cantilever wall, embedded in the ground or in an embankment that rotates when loaded and is thereby stabilized by reactive lateral earth pressures. Lessons learned from Hurricane Katrina indicate that formation of a flood-side gap between the sheet piling and the foundation soils can contribute to poor performance of I-walls in global stability, may increase seepage and uplift problems, and can increase lateral loads on the I-wall requiring greater piling tip penetration to ensure stability. These conditions lead to failure modes that should be included during design. Design and evaluation of I-walls are covered in EC 1110-2-6066 and ETL 1110-2-575.

Depending on the application, I-walls are subject to varying global stability failure mechanisms. Failure of an I-wall and embankment slope towards a river or canal may be a concern during low water levels, but failure toward

the protected side must be considered during high water levels. Global stability analyses should be conducted without using a gap between sheet piling and soil. When I-wall/earthen embankment composite systems are proposed, global stability of the levee and riverbank must also be addressed. Sheet piling associated with I-walls can act as reinforcement within the embankment and enhance global stability. Until further research is done to quantify the additional stresses imposed on I-wall sheet piling within embankments, designs for embankment portions of I-wall/earthen embankment composite systems should follow in guidance from EM 1110-2-1913, excluding any reinforcing effect of the sheet piling.

Rotation failure due to inadequate piling penetration also is possible. Classical earth pressure theories are typically used to estimate lateral earth loads and required piling penetration for cantilever walls. This type of failure is prevented by adequate penetration of the piling for the cantilever wall. If a gap forms on the flood side, then both the force and resistance are reduced and therefore the overturning moment increases, not decreases.

The loads governing the design of an I-wall arise primarily from the water loads applied to the I-wall stem, buried sheet piling, and foundation soils. Other loads applied to I-wall systems include impact, ice, and wind forces. Current methodologies for determination of these loads are discussed in EM 1110-2-2504 and ETL 1110-575. The most recent earthquake guidance is given in ER 1110-2-1806.

The Onset of Overtopping loading condition represents a rising river with the water elevation at or above the top of the wall. The water level or saturation level on the protected side should be based on project specific hydrology and hydraulics, and existing interior drainage features and projected overtopping flows, but will likely be at the top of ground. This loading condition is usually the maximum differential loading condition.

The Design Overtopping Level loading condition is a resilience and/or toughness analysis and corresponds to a water level at or above the top of the wall. The Design Overtopping Level water level is based on projected overtopping flows during unusual or extreme events. The water level or saturation level on the protected side should be based on project-specific hydrology and hydraulics, and existing interior drainage features and projected overtopping flows, and will likely be above the top of ground.

EM 1110-2-1902 provides criteria to be used with methods of stability analysis that satisfy all conditions of static equilibrium for flood walls. Finite element analyses may also be used to solve for global stability. Rotational stability is satisfied when minimum required safety factors are applied to Mohr-Coulomb shear strength properties prior to analyzing tip penetration. The use of effective shear strength properties is discussed in Chapter 5 of EM 1110-2-2504. Projects with past seepage erosion concerns should be analyzed on an individual basis relating past to expected performance. The selection and application of material properties for analyzing the stability of walls and slopes is detailed in EM 1110-2-1902 and EM 1110-2-1913. Failure towards the flood side also is covered in these two engineer manuals.

3.2.3 System components

Riverine flood-protection systems typically contain many component structures in addition to levees and floodwalls. These include gravity drainage systems, pumping stations, closure structures, and other features. They are not covered in detail in this document, but their design, maintenance, and operation during flood events is critical to the function of the flood-protection system. Closure structures, in particular, must be tested periodically, and personnel must be available for operation during a flood event. Design guidance for closure structures for openings in earthen embankments and flood walls of inland, local flood-protection projects is located in EM 1110-2-2705. Closure structures are required at openings in earthen embankment and floodwall systems when facilities such as railroads, roadways, and pedestrian walkways pass through earthen embankment and floodwall systems at elevations below the level of protection provided by the project. Closure structures for openings in earthen embankment and floodwall systems include various gate mechanisms such as stop logs (made of aluminum or steel), swing gates, miter gates, trolley gates, and various types of rolling gates.

3.2.4 Riverine and other noncoastal considerations

The performance of riverine flood-protection systems may be compromised by river processes. Channel migration or incision may erode earthen embankments. Channel sedimentation (deposition or aggradation) may cause higher stages than designed; this may result in embankment overtopping at a more frequent event than designed, or it may result in overtopping at an unplanned location. River morphology and sedimentation

processes need to be considered in design and also need to be evaluated during the life of the project. Changes are often gradual and may take place at a distance from the embankment footprint, resulting in changes to the water surface profile that are not evident until a flood event occurs. EM 1110-2-1418 provides guidance.

Ice effects are addressed in several references. Current USACE guidance on the development of ice-affected stage frequency relationships is covered in ETL 1110-2-576. EM 1110-2-1612 is a comprehensive reference covering every aspect of ice-related design and analysis. *Design-Basis Flood Estimation at Nuclear Plants in the United States of America* (NRC 2011) discusses mechanisms of ice-induced flooding.

4 Incorporated (Secondary) Barriers

Incorporated protection is provided by special design of walls and penetration closures. Walls are usually reinforced concrete designed to resist the static and dynamic forces of the DBFL and incorporate special waterstops at construction joints to prevent leakage. Penetrations include personnel access, equipment access, and through-wall piping. Pipe penetrations are usually sealed with rubber boots and flanges. Personnel access closures include submarine doors and hatches. Penetrations that are too large to close with a single door generally require stop logs or flood panels for closure. The maritime industry should be consulted for detailed information concerning the closure structures for incorporated barriers.

Design, construction, performance, and reliability standards of incorporated barriers are limited. The National Flood Barrier Testing and Certification Program (http://nationalfloodbarrier.org) is implementing a national program of testing and certifying flood barrier products used for flood proofing and flood fighting. This program currently tests barrier products in two broad categories: temporary flood barriers and closure devices. The purpose of the program is to provide an unbiased process of evaluating products in terms of resistance to water forces, material properties, and consistency of product manufacturing. This is accomplished by testing the product against water-related forces in a laboratory setting, testing the product against material forces in a laboratory setting, and periodic inspection of the product manufacturing process for consistency of product relative to the particular product that received the original water and material testing. Upon products meeting the consistency of manufacturing criteria and meeting the established standards for the material and water testing, the certification part of the program becomes available to the product.

The USACE does not have design guidance on the closure structures for secondary barriers. However, design guidance for closure structures for openings in earthen embankments and flood walls of inland, local flood-protection projects is located in EM 1110-2-2705. Examples of flood-proofed structures in the United States are included in *Flood Proofing Systems and Techniques* (U.S. Army Corps of Engineers (USACE) 1984). This document notes that plastic, marine paints, water-proofing

compounds, and other sealants can be applied to structures, but it is extremely difficult to make closures completely watertight, and many systems using this technique employ pumps to evacuate leakage.

There are two types of flood proofing of incorporated barriers: wet flood proofing and dry flood proofing. The first technique allows floodwater to enter the structure. Vulnerable items such as utilities appliances and furnaces are relocated or waterproofed to higher locations. By allowing floodwater to enter the structure, hydrostatic forces on the inside and outside of the structure can be equalized, reducing the risk of structural damage. The second technique is known as dry flood proofing. With the dry flood proofing technique, a building is sealed so that floodwaters cannot get inside. The intent of a nuclear power plant incorporated barrier is to be a dry barrier with an internal system of drainage and pumping for leakage inside the incorporated barrier. Dry flood proofing is applicable in areas of shallow, low-velocity flooding. All areas below the flood-protection level are made watertight. Walls are coated with waterproofing compounds or impermeable sheeting. Openings such as doors, windows, sewer lines, and vents are closed with permanent closures or removable shields, sandbags, valves, etc. Dry flood proofing maximum protection level is 3 ft and is not for buildings with basements since those structures are difficult to protect from underseepage. Some of the disadvantages of this technique are that many waterproofing compounds are not made to withstand the pressures of the water and will deteriorate over time. Also, closures on windows and doorways are dependent on adequate warning time for installation, as well as the presence of someone to install them correctly.

Much research and documentation of flood proofing has been conducted and/or compiled by the USACE National Nonstructural Flood Proofing Committee (NFPC). The NFPC website (http://www.usace.army.mil/Missions/CivilWorks/ProjectPlanning/nfpc.aspx) contains flood proofing information and links to online reports, including the following, that may be applicable for a nuclear power plant:

- *Flood Proofing Tests - Tests of Materials and Systems for Flood Proofing Structures* (USACE 1988) addresses closures, materials, and systems that were tested to determine the effectiveness in protecting structures from floodwaters.
- *Flood Proofing - How to Evaluate Your Options* (USACE 1993b) is a layperson's guide to evaluating and selecting flood proofing

alternatives. It includes simplified damage, cost, and performance analyses.

- *In the Tug Fork Valley: Flood Proofing Technology* (USACE 1994) summarizes and provides technical details, photos, and information regarding one of the largest Federal, nonstructural flood proofing projects ever completed within the United States.
- EP 1165-2-314 is a source for flood proofing regulations and technical schematics depicting various structural components of flood proofing measures.
- *Flood Proofing Performance - Successes and Failures* (USACE 1998) documents the successes and failures of various nonstructural flood proofing measures from poststorm events throughout the United States.
- *Flood Proofing: Techniques, Programs and References* (USACE 2000) addresses the approaches to flood proofing and government flood proofing programs, references, and terminology. It presents a general overview of flood proofing techniques and provides the reader information on government agencies that offer more specific assistance and publications containing detailed flood proofing information.

Most wall materials, except for some types of high-quality concrete, will leak unless special construction techniques are used. The most effective method of sealing a brick-faced wall would be to install a watertight seal behind the brick when the building is constructed. For flood proofing existing structures, the best way to seal a wall is to add an additional layer of brick with a seal *sandwiched* between the two layers. It is possible to apply a sealant to the outside of a brick or block wall. Cement or asphalt-based coatings are the most effective materials for sealing a brick wall while clear coatings such as epoxies and polyurethanes tend to be less effective. As a result, the aesthetic advantages of a brick wall are lost with the use of better sealant coatings.

Personnel-access watertight doors are very similar to sliding or hinged flood shields in purpose, yet they are designed to function as actual doors that are used during normal operating conditions. This type of door can be closed and sealed by a latch mechanism without the use of bolts that are normally used to secure a flood shield. These doors must be capable of resisting flood-related forces. These are the forces that would be exerted upon the building as a result of floodwaters reaching the DBFL and include the hydrostatic force, buoyancy, hydrodynamic and backflow force, and debris impact

forces (FEMA 1993). The building's utilities and sanitary facilities, including heating, air conditioning, electrical, water supply, and sanitary sewage services must be located above the DBFL, completely enclosed within the building's watertight walls or made watertight and capable of resisting damage during flood conditions (FEMA 1993).

Design guidance for closure structures for openings in levees and flood walls of inland local flood-protection projects is located in EM 1110-2-2705. Closure structures are required at openings in levee and floodwall systems where facilities such as railroads, roadways, and pedestrian walkways pass through levee and floodwall systems at elevations below the level of protection provided by the project. Closure structures for openings in levee and floodwall systems are usually either stop-log or gate-type closures.

Currently, adequate data and analyses do not exist in order for the USACE to recommend the use of incorporated barriers as a reliable flood-protection barrier at a nuclear power plant. Incorporated barriers may be able to supplement a complete flood-protection strategy, but without adequate supporting data, should be considered insufficiently reliable as a part of the complete flood-protection strategy.

4.1 Mechanical or electrical system penetrations

Although the USACE expertise in mechanical and electrical systems in flood-protection systems is primarily through dams and levees, the principles generally apply to using mechanical and electrical systems in incorporated barriers. This chapter summarizes applicable information that can be found in *Best Practices in Dam and Levee Safety Risk Analysis* (Bureau of Reclamation and U.S. Army Corps of Engineers 2012).

To control operation of an item manipulated by mechanical or electrical systems in an incorporated barrier, three things must be provided: power to move the item, machinery to operate the item, and the structural item itself. Before evaluation of the risk and reliability of operating the item, various components that make up the system and the probability of each component's failure must be defined.

There are multiple failure models that allow the user to model a component or system that undergoes periodic inspection but is also subject to aging (i.e., the failure rate increases with time). These models

also can represent a component whose failure will be revealed due to periodic usage during normal operations. *Best Practices in Dam and Levee Safety Risk Analysis* (Bureau of Reclamation and USACE. 2012) contains details and analysis demonstrating the use of a few of these models in defining the risk and reliability of structures utilizing mechanical and electrical systems.

4.1.1 Utilities

Deteriorated culverts, pipes, and utility lines below the foundation of flood barriers may result in underseepage and piping, compromising the structural foundation. This is particularly true for pipes that are constructed of materials likely to degrade over time and are not routinely inspected to determine their actual condition. A defect through a pipe below a flood barrier such as a floodwall can lead to a preferred seepage path and depending upon the conditions (surrounding soil, loading duration, and etc.), can cause piping of foundation materials through the defect. This can cause a loss of foundation support, wall instability, and failure of the structure. Defects can occur either through the body of the pipe (perforations) or at separated joints. A thorough review of the as-built plans and permits needs to be done in order to determine if and where pipes cross below project flood barriers. Deteriorated pipes running parallel to flood barriers can also be an issue if they are close enough to adversely affect performance from an underseepage or stability standpoint. The *Levee Screening Tool Technical Manual* (USACE 2010, draft) provides a detailed narrative on adverse environments for various types of pipes and is a good resource to determine if a pipe may affect flood-protection performance. Sewer lines should be fitted with cutoff or check valves that close when flood waters rise in the sewer to prevent backup and flooding inside the building. Emergency power is vital to the operation of a flood-protection system for a nuclear power plant. Every aspect of the system must have protection, including protecting the fuel used to operate emergency generators.

4.2 Personnel penetrations

Dry flood proofing involves sealing building walls with waterproofing compounds, impermeable sheeting, or other materials and using shields for covering and protecting openings from floodwaters. In areas of shallow, low-velocity flooding, shields can be used on doors, windows, vents, and other building openings. The first step with the use of shields placed

directly on buildings is to be certain that both the shield and the building are strong enough and sufficiently watertight to withstand flood forces. Generally, dry flood proofing should only be employed on buildings constructed of concrete block or brick veneer on a wood frame. Even brick or concrete block walls should not be flood proofed above a height of 3 ft, due to the danger of structural failure from hydrostatic forces.

Some waterproofing compounds cannot withstand significant water pressure or may deteriorate over time. For effective dry flood proofing, a good interior drainage system must be provided to collect the water that leaks through the sealant or sheeting and around the shields. These systems can range from small wet-vacs to a group of collection drains running to a central point from which water is removed by a sump pump. In many cases, flooded sites are isolated during a flood event. Once barriers, shields, closures, etc., are installed at a site, the normal site egress paths are often obstructed or removed. Attention must be given to this safety issue.

The difficulty and complexity of sealing a structure also depends on the type of foundation, since all structural joints, such as those where the walls meet foundations or slabs, require treatment. For very low flood levels, such as a few inches of water, a door can be flood proofed by installing a waterproof gasket and reinforcing the door jamb, hinge points, and latch or lockset and coating it with a waterproof paint or sealant. If there is a chance of higher flood levels, some type of shield will be needed. If the expanse across the door is 3 ft or greater, the shield will have to be constructed of heavy materials, such as heavy aluminum or steel plate. The resulting weight may require the shield to be permanently installed, using either a hinged or slide-in design. The frame for such an installation must be securely anchored into the structure. When windows are exposed to flooding, some form of protection is needed because standard plate glass cannot withstand flood forces. One solution is to brick up all or part of the window. It may also be possible to use glass block, instead of brick, to admit light.

For normal-sized windows, shields can also be used. They should be made of materials such as heavy Plexiglas, aluminum, or framed exterior plywood. These can be screwed into place or slid into predesigned frame slots. Another alternative is to replace the glass with heavy Plexiglas;

however, the window must be sealed shut and waterproofed using water-resistant caulking.

The specifications for materials, fabrication, installation, and quality assurance of the various components of closure structures are provided in the *Unified Facilities Guide Specifications for Waterway and Marine Construction* (USACE 2008). More detailed information is available in the Unified Facilities Guide Specifications library (http://www.wbdg.org/ccb/browse_cat.php?c=3).

4.3 Warning systems

Warning systems may be able to reduce the risk of danger to people and structures. However, the warning system itself is subject to potential operational failure. The following steps are required for a warning system is to be successful:

1. Failure is detected by the system and the alarms are triggered.
2. A decision is made to initiate an action.
3. The population at risk is notified of the impending failure.
4. The population at risk is successfully prepared prior to the failure.

The following two factors make a warning system *more likely* to detect a flood failure:

1. There are multiple independent platforms to collect and transmit data. This provides redundancy in transmitting data.
2. There are numerous independent instruments that provide for possible verification of a flood failure.

The following two factors would make the warning system *less likely* to detect a flood failure:

1. A false alarm has already occurred. The instrumentation is not 100% reliable.
2. A major seismic event near the site capable of failing the flood control structures could wipe out all communications platforms at the site. While this could be an indication of a flood failure, it could also be interpreted as something else.

Given a flood failure and that the early warning system successfully detects the failure through the alarm systems in place, two factors that make the decision to initiate an action more likely include the following:

1. Operating personnel have taken part in an exercise related to flood failure and the need to secure facilities and staff.
2. Operating personnel have been given the authority to initiate the action. The notice to begin the action can be given directly without going through other offices for approval.

5 Temporary Barriers

Flood protection for critical infrastructure should not rely on temporary structures, and permanent structures capable of withstanding floods of a desired probability should be constructed. However, in cases of an extreme event or combination of events that threaten to overtop the primary flood defenses, temporary measures may be deployed. Traditionally, sandbags have been used as temporary flood barriers to raise the effective crest of a levee or as a barrier encircling a building(s). Numerous commercial products are available that are capable of raising the effective crest of an earthen barrier on a temporary basis in far less time than is required if sandbags are used. Most of these temporary barriers may be classified as sand-filled, water-filled, frame-with-skirt, among others. Temporary flood-protection structures, without adequate supporting data, should be considered insufficiently reliable to be included in a complete flood-protection system. However, the use of temporary barriers can supplement a complete flood-protection system of external barriers, an interior drainage system, and redundant pumping stations.

The USACE has a Flood Fighting Structures Demonstration and Evaluation Program intended to devise real-world testing procedures for promising flood-fighting technologies (http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=PROGRAMS;16). As part of this program, in 2004 the ERDC tested a few temporary, barrier-type, flood-fighting structures. This chapter summarizes the testing and evaluation of sandbags as well as three commercial flood-fighting products. The full analysis can be found in ERDC TR-07-3 *Flood-Fighting Structures Demonstration and Evaluation Program: Laboratory and Field Testing in Vicksburg, Mississippi* (USACE 2007).

Laboratory and field testing were conducted from March to August 2004. The laboratory testing was completed in a wave research basin at ERDC, Vicksburg, MS. Field testing was accomplished at a site north of Vicksburg, on the southern bank of the turning basin of the Vicksburg Harbor. Summary results for the laboratory test are shown in Table 4-1 and for the field test in Table 4-2. Both the laboratory and field testing show conclusively that a Portadam, Hesco Bastion and Rapid Deployment Flood Wall (RDFW) structure can be constructed much faster and with much less labor force than a comparable sandbag structure. All three products performed well for most of the testing parameters.

Table 4-1. Laboratory test summary.

Item	Portadam	Hesco Bastion	Sandbags	RDFW
Construction time (hours)	4.8	3.5	11.5	5.5
Construction effort (man hours)	24.4	20.8	205.1	32.8
Removal time (hours)	1.1	2.7	4.5	7.0
Removal effort (man hours)	4.4	13.4	9	42
Seepage (gallons/minute/ft) for static water at 95% of structure height (average)	0.14	1.81	0.54	0.10
Damage from overtopping	None in 1 hour (hr)	No damage in 1 hr	Failed in 1 hr	None in 1 hr
Damage from log impact	Vinyl tarp puncture	No damage	No damage	No damage
Repairs concern	Minor	Minor	Major	Minor

Table 4-2. Field test summary.

Item	Portadam	Hesco Bastion	Sandbags	RDFW
Construction time (hours)	5.1	8.9	30.5	7.5
Construction effort (man hours)	26.2	57.5	453.1	48.4
Removal time (hours)	2.9	8.7	2.6	17.3
Removal effort (man hours)	12.6	36.3	3.5	113.4
Seepage (gallons/hr) for 400 ft^2 wetted area	550	6,000	300	900
Repairs	Minor	Minor	Minor	Minor
Reusability (%)	100	> 95	0	> 90

5.1 Sandbags

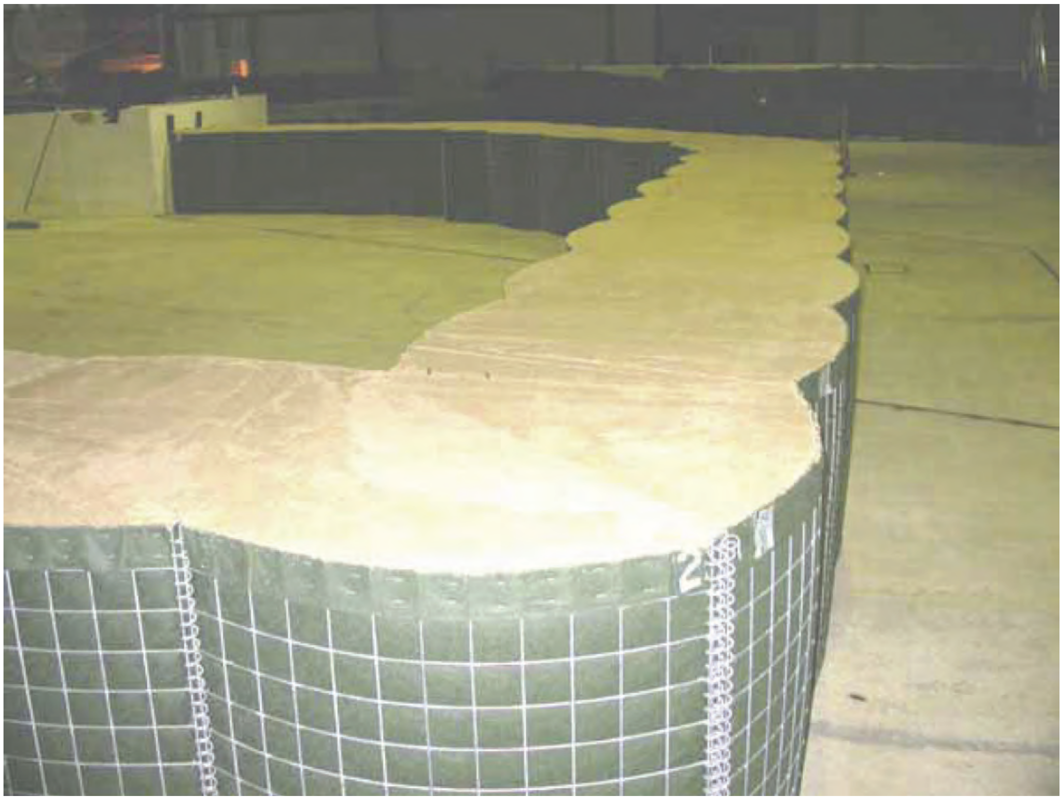
Sandbag barriers have traditionally been the method of choice to raise earthen embankment heights and to protect infrastructure against rising floodwaters. Sandbag levee construction protocol calls for a width three times that of the height as the minimum width criteria. The strengths of a sandbag structure include low product cost. Sandbags also conform well to varying terrain. In both the laboratory and field tests, the sandbag structure had low seepage rates. Also, sandbag structures can be raised if needed by simply placing additional sandbags. The weaknesses of a sandbag structure are that they are labor intensive and time consuming to construct. Also, sandbags are not reusable. During the laboratory testing (Figure 5-1), the sandbag structure was damaged during the wave impact tests and failed during the overtopping tests. The sandbags began to deteriorate during the field tests. For additional information on sandbags, materials, tools, and equipment, see section VI of the *USACE Flood Fight Manual* (USACE 2010).

Figure 5-1. Laboratory sandbag configuration (from U.S. Army Corps of Engineers, 2007).

5.2 Hesco Bastion Concertainer

Hesco Bastion's strengths include ease of construction and removal for both time and manpower. These structures were constructed much faster and with much less labor force than the sandbag structures. The Hesco Bastion product is relatively low cost, and the structure can be raised if required by placing a second row of units to the top of the structure. Stability can become an issue for increased height due to the narrow width of the Hesco units. If stability is an issue, a pyramid structure should be constructed (two units wide on bottom row topped with a single row of units). Hesco Bastion units proved to have a high degree of reusability. During the laboratory and field testing, the structures suffered only minimal damage (Figure 5-2). Weaknesses of the Hesco Bastion product include the need for significant right of way due to the addition of granular fill with machinery perpendicular to the structure and high seepage rates. Since completion of the testing, Hesco Bastion has evaluated their high seepage rates. Their evaluation concluded that in both the laboratory and field testing, the units were installed incorrectly. When retested in the laboratory, the seepage rate for a properly installed barrier at a depth of 1 ft was reduced by 75% from the original test results.

Figure 5-2. Laboratory Hesco Bastion Concertainer configuration (USACE 2007).



5.3 Geocell Systems, Inc., RDFW

Geocell Systems RDFW's strengths include ease of construction for both time and manpower. In both the laboratory and field testing, the RDFW structures were constructed much faster and with a much smaller labor force than the sandbag structures (Figure 5-3). Additional strengths of the RDFW structures included low seepage rates and high degree of reusability; an RDFW structure can be raised as needed by placing additional rows of units to an existing structure. Since the RDFW units are 8 inches (in.) high, an RDFW structure provides various height options. For instance, if a user purchased a quantity of RDFW to construct a 4 ft high flood-fighting structure 1,000 ft long and in a particular flood only needed a 2 ft high structure, then this user would have sufficient product to construct a 2,000 ft long structure. RDFW's weaknesses include significant right of way required due to the placement of granular fill with machinery perpendicular to the structure, high cost of the product, and in both the laboratory and field testing, the RDFW structures were difficult and time consuming to remove. Although the units were easy to fill with clean granular material during the tests, the small size of the grid openings (8 in. by 8 in.) may be a problem for construction with unsorted local fill material.

Figure 5-3. Laboratory Rapid Deployment Flood Wall (USACE 2007).

5.4 Portadam

Portadam's strengths include ease of construction and removal (time, manpower, and equipment). The Portadam structures were constructed in less time and with a much smaller labor force than the sandbag structures. Also, the Portadam structure was constructed without the use of heavy machinery and proved easy to remove. The Portadam structure had low seepage rates in both the laboratory and field tests (Figure 5-4). These structures require no fill except for some sandbags that are used to help seal the leading edge of the membrane liner and to add weight to prevent wind damage. Portadam structures have a high degree of reusability; for the field test, the structure was 100% reusable. Since no heavy machinery is required to construct a Portadam structure, only limited right of way is required. However, this structure does have the largest footprint of the products tested. Portadam's weaknesses include that the membrane liner punctured during the laboratory debris impact tests and that a Portadam structure may not be applicable for high-wind use unless the structure is anchored or weighted with sandbags. Additionally, it is not possible to raise the height of this barrier once erected, if additional height becomes necessary.

Figure 5-4. Laboratory Portadam configuration (USACE 2007).

5.5 Generalizations

Although the government-funded study described above ended in 2007, ERDC has maintained the laboratory testing basin and continues to test temporary flood-fighting products using the same standardized testing protocol on a vendor-funded basis. Based on these tests, the following generalizations are offered.

Most of the sand-filled barriers, such as the Hesco-Bastion Concertainer (section 5.2) and Geocells System RDFW (section 5.3) that were tested were developed as force protection measures designed to stop bullets rather than hold back floodwaters but are being deployed as a replacement for sandbags to raise the effective crest elevation of earthen barriers. Various products were tested at water depths up to approximately 3 ft and generally performed well with moderate seepage rates. Products tested ranged from low-cost disposable geotextile units to higher-priced reusable units with frameworks of plastic, sheet metal, or wire grid.

Failure modes of sand-filled barriers include overturning, sliding, undermining, loss of fill material, and damage to the structure framework. Overturning may become a problem if the barrier is built multiple layers in height without widening the base of the structure or when the structure is undermined either on the riverside or protected side. Protection against overturning may be possible by using multiple rows of units to make a wide base, then fewer rows on top, like a pyramid. Sliding has not been

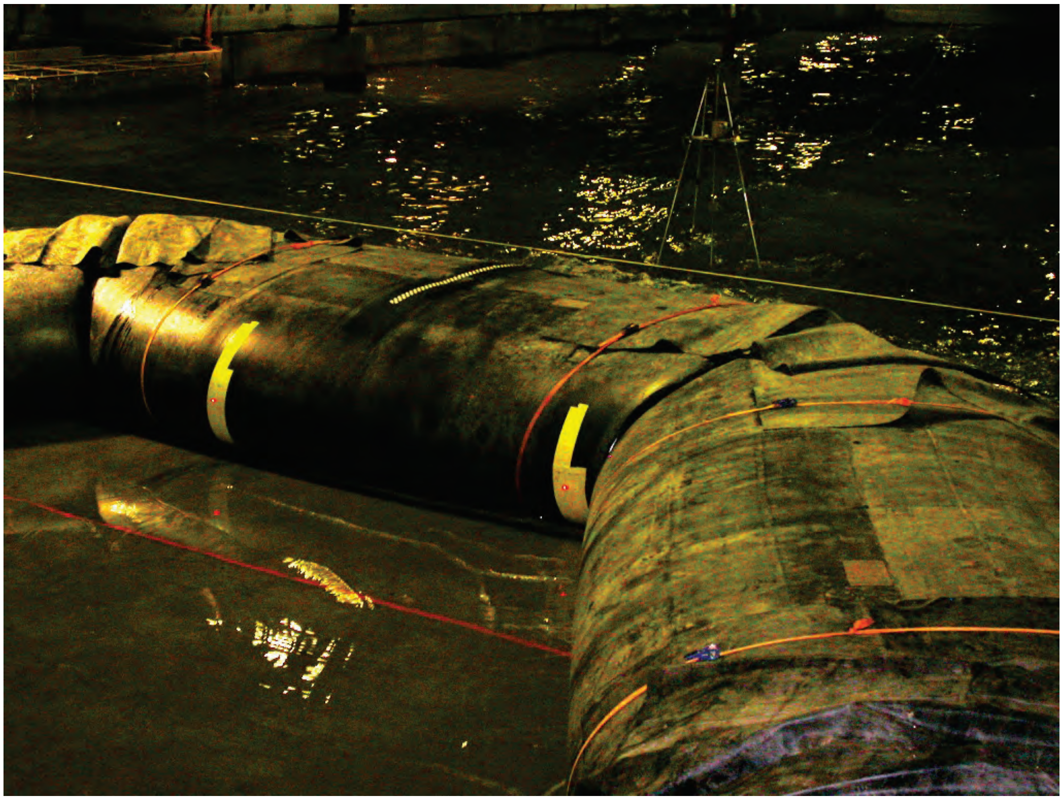
found to be a problem in the laboratory tests but has been shown to be a concern with barriers built on frozen ground. Undermining is not a problem with static water but may be an issue with waves or currents undermining the river-side toe of the barrier or overtopping or seepage flow undermining the protected toe. Undermining may lead to overturning of the structure or, if the structural units are open bottomed, to loss of fill material. Fill material may also be lost from the top of the units due to wave action or overtopping or from the bottom of open-bottomed units due to rocking of the barrier during wave action causing the toe to lift and fill to be washed out. Damage to the framework may occur from debris impact or wave action causing tears in fabrics or breaks in solid barriers.

The most common maintenance issue in the laboratory tests was a need to replace sand fill that was washed out by wave action or overtopping. These temporary barriers need to be inspected regularly to correct any observed problems of leaning, sliding, or scour. If the barrier is going to be reused, it must be cleaned thoroughly, allowed to dry, and any damaged components replaced. Fabric sections, if present, should be inspected for areas weakened by ultraviolet rays.

The advantage of water-filled barriers is that there is always plenty of water during a flood, so the fill material does not need to be transported to the site. Water-filled barriers include tubes made of reinforced plastic, rubber, or other materials, or hard-shell cases (typically plastic or fiberglass) that are fastened together in a row(s) and filled with water. One type of water-filled tube is shown in Figure 5-5.

A disadvantage of water-filled tubes is that they roll easily if water is raised on one side of the barrier. Commercially available tubes for flood fighting use one of three methods to prevent rolling: internal baffles, multiple tubes fastened together with straps, or anchors. Because water-filled tubes are not round but more like a flattened oval when filled on dry land, an internal baffle spanning the short axis of the oval will prevent rolling as it stops the short axis from rolling over to the long axis. If two or more tubes lying side-by-side on the ground are fastened together with straps, friction between the tubes will prevent the barrier from rolling. By fastening multiple barriers together in a pyramid shape, it is also possible to raise the height of the barrier. The third option is to fasten a tube to ground anchors on the riverside of the barrier for static water or currents or on both sides of the barrier if wave action is expected.

Figure 5-5. A Floodwall water-filled tube being tested in the laboratory in 2007.



A second disadvantage of water-filled barriers is that the water fill has no weight when submersed. The water-filled barrier gets its stability by the weight of the water inside the barrier that is higher than the outside water level. Generally, water-filled barriers should not be used if the river depth at the structure toe is greater than two-thirds the height of the barrier. At deeper water levels, the tubes tend to not seal well along their bottom, seepage rates increase, and the barrier may become unstable.

Failure modes for water-filled barriers include rolling, undermining, punctures, and leaking. Rolling was discussed above. Undermining may be an issue with waves or currents undermining the riverside toe of the barrier, overtopping may lead to undermining of the protected side toe, or seepage flow under the barrier may cause undermining. An advantage of a flexible barrier such as a fabric tube is that it may flex into a scour hole or area of undermining and prevent the hole from enlarging. Punctures caused by debris impact, vandalism, or construction mishaps are serious in that an entire unit may drain from a single hole unless it is repaired. Because a tube may be 100 ft or more in length, draining an entire tube could cause a serious breach in the barrier. Leaking at seams or ports may also drain a unit, lowering the crest elevation and weakening the barrier.

The most common maintenance issue for water-filled barriers is repair of leaks. Other issues include repair of scouring or undermining, sliding, and separation at joints between adjacent units. As most water-filled barriers are reusable, cleaning and drying the units for storage is critical. If the units were filled with water from the flooding river, there may be mud and debris inside the tube that can be difficult to remove.

Frame and skirt barriers, such as the Portadam (section 5.4) consist of a sheet of plastic or other flexible material that is fastened to a framework to provide elevation to the barrier while the other end of the sheet is spread out on the ground on the riverside of the barrier and extends outward several feet or tens of feet from the barrier as a skirt. The outer toe of the skirt may be trenched or held in place with sandbags. As the flood waters rise up over the skirt, the weight of the water holds the skirt against the ground, providing a seal.

Failure modes include tearing of the fabric, seepage under the skirt, or failure of the frame. The fabric draped over the framework may be cut by water-borne debris, vandalism, or installation errors. The skirt can be partially lifted by wave action, currents, or debris, which can lead to water flowing under the barrier. Properly designed and installed, the frame should be able to withstand static water pressures but may be damaged by debris impact or wave action. Settling of the frame in mud or soft soil may be a problem.

Maintenance issues include repair of tears in the fabric and repair of damaged frame components. Issues with settling or sliding may need to be corrected. It may be necessary to add sand bags to the skirt to improve the seal or keep it from being moved by wind (preflood) or waves and currents. Most frame and skirt barriers are reusable, so the units must be thoroughly cleaned, dried, inspected, and repaired before returning to storage.

There are many other types of temporary barriers, including interconnecting blocks of various shapes, impervious panels in different configurations, gated structures, self-rising flood barriers, and more. Some require preinstallation of a portion of the barrier such as a base or anchoring system (*semipermanent* flood barriers). All types have their own advantages and disadvantages, storage requirements, and installation and removal requirements. Summaries of tests conducted by the USACE may be found at http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=Projects;182 .

6 Locally Intense Precipitation

While many nuclear power plants face external flood mechanisms such as nearby flooding rivers or storm surge in a coastal area, locally intense precipitation can cause flooding inside primary barriers. If not properly accounted for and managed, locally intense precipitation can turn into a significant flooding threat. For example, recent hurricanes and major named storms have produced 10–20 in. of rainfall over significant areas in just a few hours. Properly designed and maintained interior drainage systems and pumping stations alone are insufficient for flood protection at nuclear power facilities. However, coupled with properly designed and maintained external barriers, the combined flood-protection system is very reliable.

Interior drainage in a local flood-protection project is caused by local precipitation, seepage from temporary flood-fighting structures, levees, floodwalls, and other flood-protection structures. EM 110-2-1413 provides an overview of the concepts and strategy for examining the potential for interior flooding. The two concerns for managing locally intense precipitation are conveyance and containment. Conveyance includes interior drainage systems, channels, pipes, culverts, and floodwall-closing structures. Containment includes structures such as outlet works, sump works, and pumping stations.

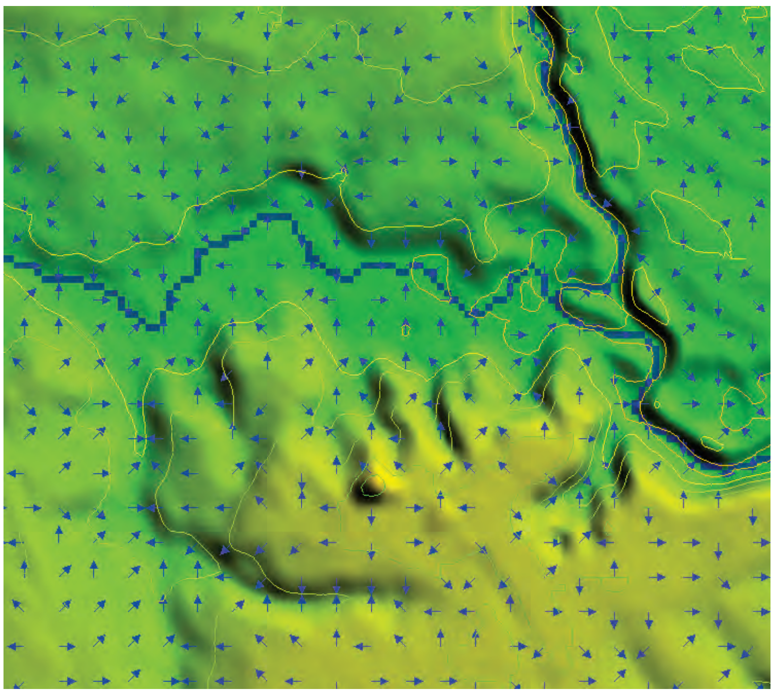
The probable maximum flood (PMF) has flood characteristics of peak discharge, volume, and hydrograph shape that are considered to be the most severe reasonably possible at a particular location, based on relatively comprehensive hydrometeorological analyses of critical runoff-producing precipitation, snow melt, and hydrologic factors favorable for maximum flood runoff. The PMF load condition represents the most severe hydraulic condition, but because of possible overtopping effects, it may not represent the most severe structural loading condition, which is represented by the onset of overtopping. Therefore, the PMF condition will not necessarily be examined for design.

6.1 Interior drainage systems

In designing a conveyance system to protect critical facilities one must understand the flow patterns in both the natural topography and the

engineered topography. This requires accurate topographic data as depicted in Figure 6-1. EM 1110-1-1005 discusses accuracy requirements, reference systems, survey procedures, and other key points related to producing accurate topographic data. There are publicly available sources of topographic data, such as the U.S. Geological Survey (USGS) National Map topographic data (http://www.nationalmap.gov), but care should be taken to ensure that products used have sufficient vertical accuracy for use in critical protection efforts.

Figure 6-1. Example natural flow directions, contours, and streams from publicly available topographic data.



Interior drainage in a local flood-protection project is caused by local precipitation, seepage from temporary flood-fighting structures, levees, floodwalls and other flood-protection structures. EM 110-2-1413 provides an overview of the concepts and strategy for examining the potential for interior flooding.

6.2 Conveyance structures: Channels, pipes, and culverts

Natural and engineered structures work as a system to convey rainfall. Storm drainage conveyance structures include channels, culverts, pumps, and pipe networks. These structures should be designed to convey the required water quantities as well as to not be subject to erosion or deposition or debris blockage that would degrade the conveyance or

reduce the capacity. EM 1110-2-1205, EM 1110-2-1418, and EM 1110-2-1601 can be consulted for design guidance, ensuring its long-term stability and functional operation during flood-fighting situations.

EM 1110-2-2902 covers the design and application of pipes through levees to create outflow mechanisms. EM 1110-3-136 discusses designing pipe networks, culverts, inlets, and other internal drainage conveyance structures. Pipe networks that operate as outlet works through a levee or floodwall should have appropriate closing mechanisms to prevent backflow.

6.2.1 Streambank protection

Designs of conveyance and containment structures for the long-term protection of critical structures must account for the effects of changing hydraulics on the natural system. In the natural environment, streams operate as sediment conveyor belts. This sediment conveyor belt system creates a dynamic equilibrium with the channel shape, slope, and bed material. Engineered channels that are underdesigned for the sediment transport requirements of the natural channel will develop sedimentation problems that will significantly reduce the capacity of the channel and hence the protective ability of the channel. Engineered channels that are overdesigned for sediment transport will increase the erosion potential either in the channel or downstream of the engineered portion of the channel. Erosion can cause flood-protection structures to fail, or it can alter the course of the channel towards an undesired location, resulting in an increased threat to the facility.

The WES Stream Investigation and Streambank Stabilization Manual (USACE 1997) discusses the geomorphologic and hydraulic aspects of channel design for streambank protection, various structural and nonstructural protection mechanisms, lifecycle management of the engineered channel, as well as maintenance and monitoring requirements for engineered channels.

6.3 Floodwall closing structures

Frequently floodwalls have openings that facilitate transfer of vehicles, materials, and personnel in and out of the contained area. These breaks in the flood defenses need to have closing structures that perform to the required level of reliability. EM 1110-2-2705 discusses various flood-gate mechanisms, including stop logs (made of aluminum or steel), swing gates, miter gates, trolley gates, and various types of rolling gates.

6.4 Containment structures

During flood fights that include interior drainage as well as high water on the outside of the containment area, the locally intense precipitation must be contained or pumped beyond primary barriers. Containment systems include sump basins and detention basins. EM 1110-2-1420 discusses the role and design principles of reservoirs and detention basins. Design guidance for detention basins of a permanent nature can be found in EM 1110-2-2300. The design capacity of the basin or sump should be for a rainfall event that takes into consideration the frequency of rainfall along with the associated (correlated) frequency of high water precluding discharge of the basin. The volume calculations also need to include seepage through other flood-fighting defenses.

6.4.1 Outlet works

Outlet works serve to regulate or release water impounded by a dam or detention basin. It may release incoming flows at a reduced rate, as in the case of a detention dam; divert inflows into canals or pipelines, as in the case of a diversion dam; or release stored water at such rates as may be dictated by downstream needs, evacuation considerations, or a combination of multiple-purpose requirements. EM 1110-2-1420 describes spillways and outlet works concepts for reservoir design.

In reservoir design, there are two purposes of the outlet works. The first is to control the outflow, and the second is to prevent the rapid failure of the dam by overtopping. Designing these two levels of outlet works requires an understanding of the potential inflows and total volume of rainfall to be impounded. The design of the regular-use spillways is generally dependent upon the flow rates expected to be input to the system during regular and emergency events. The design of the outlet works also calls for an emergency outlet that is sufficient to pass (not contain) the PMF in order to prevent the immediate failure of the dam from erosion due to overtopping flows. EM 1110-2-2400 covers the design of outlet works, including seismic requirements and various gates, valves, and other equipment.

6.4.2 Sump works

The American National Standards Institute/Hydraulic Institute (ANSI/HI) Pump Standards (http://www.pumps.org/content_detail.aspx?id=1412) is the primary source for current information on pump standards and design.

The ANSI/HI Pump Standards includes definitions, industry terminology, design, application, installation, operation and maintenance guidelines, plus the widely accepted Hydraulic Institute Test Standards.

The USACE has several manuals related to sump works, but when the same content from these manuals is covered in the ANSI/HI Pump Standards, the general practice is to use the ANSI/HI Pump Standards. EM 1110-2-3102 discusses the design of pumping stations, electrical power requirements, and equipment selection. Structural and architectural design of pumping stations are described in EM 1110-2-3104. Mechanical and electrical design of pumping stations is described in EM 1110-2-3105. Pump outlets must be designed to avoid damage to containment barriers. For any nondry site, a reliable pumping system with built-in redundancy is recommended.

7 USACE Flood Fighting Methods

The USACE has decades of experience fighting floods. The methods developed from these experiences are documented in detail in the *U.S. Army Corps of Engineers Emergency Flood Fight Training Manual* (USACE 2010). This chapter summarizes the principal methods from that manual, associated with maintenance and repair of levees and floodwalls during a flood event. The exact steps described herein may not apply to all flood fight situations, and site-specific maintenance and flood fight plans should be developed that consider the issues discussed herein. The USACE routinely conducts flood fight exercises to ensure that personnel are properly trained and that required resources are available for actual flood fights.

A plan for maintenance activities should be developed well in advance of the normal flood season and updated as necessary to account for changes in personnel responsible for maintenance activities. The plan should include an adequate warning system and a well thought-out evacuation plan to be followed should the need arise during the life of the project. Upon receipt of official information forecasting the possibility of high water, the facility staff should immediately mobilize a skeleton organization capable of rapid expansion. Definite reaches of the project should be assigned to individuals (section leaders). Each section leader should immediately go over the entire assigned project reach and make a detailed inspection giving special attention to the following:

1. Section limits: Ascertain that dividing lines among section responsibilities are clearly defined and, if necessary, marked.
2. Condition of the drainage ditches, levees, and any recent repairs.
3. Condition of drainage structures with special attention given to flap gates.
4. Water conditions and any accumulations of trash, debris, ice, etc.
5. Transportation facilities: Vehicular roads and access.
6. Material supply: Location, item, quantity and conditions.
7. Communications: Locate and check all necessary two-way radios and telephones.

After the initial inspection, each section leader should recruit a group and perform the following work as required:

1. Fill holes, gullies, and washes in the levee crown and slopes. Farm equipment can be used in repairing small deficiencies.
2. Repair all gaps or depressions that have degraded or are lower than the original levee grade. Filling such depressions may necessitate using material from borrow pits, in which case excavation for the material should be kept at 500 ft from the toe of the levee. This type of filling should be tamped in place, and if subject to wavewash, the new section should be faced with sandbags.
3. Check all flap gates to see that they will set properly.
4. Ascertain that necessary access roads along the levee are usable or will be satisfactorily conditioned.
5. Locate necessary tools and materials (sacks, bags, brush, lumber, lights, etc.) and distribute and store same at points where active maintenance is anticipated.
6. Check all needed telephone lines for proper functioning. Obtain lists of all team forces, construction equipment, motorboats, motor cars, and truck transportation that can be made available.
7. Arrange with local citizens for supply, transportation, subsistence, and shelter for labor force.
8. Examine all drainage ditches on the landside of the levee and remove any obstructions.
9. Remove all dynamite and explosives from the vicinity of the levee.

A maintenance inspection should be made of all drainage structures any time high-water stages are forecast. No structure should be omitted from such inspection because of adequate performance during past high-water events. If any condition is found that would indicate that the flap gate will not properly operate, the gate should be trial operated at once. Most drainage structures are situated to convey interior drainage from low points of the protected area through the levee by gravity flow. Because of location, drainage structures are generally subject to inundation at lower stages than most other project features. If possible, sluice gates should be inspected before the outlet end of the structure becomes submerged, and any trash, debris, or other potential obstruction present should be removed. If the gate system provided on a drainage structure fails to operate and cannot be repaired because of high water, immediate consideration should be given to blocking the structure opening by other means.

Blocking the outlet end of the structure by sandbags is a suggested method of providing an effective temporary closure. If the efforts to plug the outlet

structure fail, immediate action should be taken to build a sandbag or earth ring around the inlet structure. While it is of the utmost importance that the structures are blocked to prevent high stages of the river from flowing into the protected area, such emergency closures should be such that they can be readily removed after high river stages recede.

An earthen levee is in potential danger whenever there is water against it. The danger increases with the height of water, the duration of the flood stage, and the intensity of either the current or wave action against the levee face. A well constructed levee of correct cross section should, if properly maintained and not overtopped, hold throughout any major flood. Potential failures due to sand boils, sinking levees, slides, or sloughing may be prevented if prompt action is taken and proper methods of treatment are employed.

7.1 Overtopping

Overtopping is the rush of flood waters over the top of the levee section. The practice of increasing the height of a levee by placing material on the crown to prevent overtopping is called capping or topping. In any high-water situation, sound practice requires that immediate consideration be given to the levee grade line. Although grade lines or profiles should be kept current, a new line of levels (survey) should be run over any reach that appears to be below the predicted flood crest. The grade, in general, is based upon a freeboard approximately 2 ft above the anticipated elevation of water. Overtopping is only one mode of failure and may not be the most important or likely mode of failure. Therefore, the concept of cliff-edge effects, “the safety consequences of a flooding event may increase sharply with a small increase in the flooding level” (NRC 2012), is not necessarily applicable.

Field supervisors should use a certain amount of judgment in determining the type and extent of capping. For example, if the profile shows that a stretch of levee requires less than 0.5 ft of capping to provide the desired 2 ft of freeboard, the capping may be temporarily omitted. If, however, 2.5 feet of capping is necessary and only 12 in. boards are available, three boards should be used although the earth needs to be built up to a height of only 2.5 ft. Since capping should be as nearly watertight as possible, care should be taken in preparing the portion of the crown of the levee upon which capping rests. All depressions, such as paths or ramps, should be restored to the natural levee grade, with adequate cross section.

Sandbags are frequently used to bring low ramps up to grade. The levee crown should be thoroughly scarified to a minimum depth of 2 in. by plowing, or other similar means, in order to obtain a watertight bond between the capping and the levee crown (levee surface). There are generally four types of capping: earth-fill, sandbag, flashboard, and mud-box or box levee.

The type of capping required is governed by local conditions. Earth-fill capping is the simplest type and quickest to construct. In areas where cohesive materials are unavailable and where the capping would not be exposed to severe wavewash, earth-filled capping can be used to a height of approximately 1.5 ft. In areas where cohesive materials (such as clay) are available, greater heights can be achieved, depending on wave action and current velocities. If the levee crown width is 20 ft or more, the height to which earth levee capping can be placed may exceed 1.5 ft.

Under usual conditions where the height capping exceeds 1.5 ft, and is less than 3 ft, or where wave action is anticipated, sandbags or flashboards can be used to raise the level of protection. Capping in excess of 3 ft in height usually requires mud-box or box-levee construction, depending on the width of the levee crown and the nature of the material used for capping. Under conditions where capping is necessary over previous high-water capping, care should be taken to provide adequate base width for the new work. The height controlling the type of structure should include the height of previous capping. That is, if the combined height of new and old capping exceeds 1.5 ft, flashboard capping should be used. In such cases, it is often necessary to resort to the mud-box or box-levee-type structure in order to provide adequate stability. The construction methods for each type of capping will be described in the order mentioned above.

The usual sources of earth-fill for capping are from the farm fields on the landside of the levee, from the banks of drainage ditches, or from the landside edge of the crown when the levee has a crown in excess of 15 ft in width. Ordinarily, material should not be taken from within 100 ft of the landside toe of the levee. It is customary to take a cut only about one spade deep over a relatively large area. The method of placing material for capping is important in order to minimize the amount of seepage through the capping. The material adjacent to the flashboards should be free from clods and stubble and should be thoroughly tamped. All additional material should be compacted as well as conditions permit.

The supervisors in charge of capping should organize crews so that the work will proceed in a regular order, each crew of people executing a particular phase of the work, such as preparing and distributing lumber, plowing, setting posts, nailing boards, placing burlap or other materials, and placing the earth fill. If long stretches of levee are to be capped to a given height in the face of a rapidly rising river, it is well to set the posts to the required height and place the bottom boards only. Succeeding boards and fill are placed after the first boards have been placed throughout. Capping work should be laid out so that the low places are concentrated on and a uniform freeboard provided, parallel to the anticipated flow line, throughout the entire length of the job. The exact method of conducting this kind of work depends upon local conditions and upon the best judgment of those in charge of the work.

These methods of capping are fairly labor intensive and costly. They are also very susceptible to wave erosion if the waves break at the intersection of the flashboard and the levee. If this case arises, protective measures should be executed to ensure that the flashboard or mud box is not undercut.

7.2 Wave wash and ice attack

The type of wave-wash protection to be constructed depends upon local conditions, whether or not the levee is exposed to severe wave wash, the materials of which the levee is constructed, the type and quantity of trees and protective vegetation which may be expected, and the existing and predicted stages of the river. The types of wavewash protection generally used are vertical board revetment, horizontal board revetment, and earth-filled sack revetment. Each of these types is described in more detail in the *U.S. Army Corps of Engineers Emergency Flood Fight Training Manual* (USACE 2010). Sometimes ice conditions are such that protection provided by the methods outlined above will not be totally effective. A boom of logs, driftwood, or any available timber fastened together, strung along the levee slope and anchored approximately 15 ft from the water's edge has proven particularly effective against ice attack.

Rock riprap is a very popular method to prevent wavewash erosion and current scour. Depending on the haul distance, this method can, however, be very costly. If site conditions and time permits, the use of filter fabric and/or bedding and spalls placed prior to riprap should be considered to prevent soil material from being pulled through the riprap layer. Straw

bales wrapped with polyethylene sheeting on the water side can be used to provide some wavewash protection. Sand bags should be used to weigh the bales down so they do not float away.

7.3 Current scour

The methods to be used in protecting a levee against current scour depend entirely upon local conditions. In some cases, the current attack is so severe and the scour is of such serious nature that it requires specially designed structures that cannot be constructed with the ordinary high-water-fighting equipment and personnel. Ordinarily however, current scour can be prevented or stopped by relatively simple techniques. The methods that can be used to prevent current scour are widening of waterway gaps in abandoned levees, protecting the riverside slope of the levee with riprap or wavewash fences, or the construction of brush dikes, each of which is described in detail in the *U.S. Army Corps of Engineers Emergency Flood Fight Training Manual* (USACE 2010).

7.4 Throughseepage

Drainage of the landside slope of the levee is one of the most important high-water maintenance operations. Consequently, the function must be fully understood and appreciated. Drainage of the adjacent terrain is also highly important. The methodology utilized in draining the slope is to concentrate the flow of seepage into directed channels that carry it rapidly down the slope and away from the levee. The result is that the slope will often become dry and firm between the drains. The drains themselves sometimes never stop flowing. Drainage alone sometimes will not stabilize a wet slope, and the slope could become unstable. If this happens, watch the slope carefully for signs of sliding or sloughing and be prepared to construct a mattress (described later) immediately.

Water seeping through a levee may first appear as a wet spot on the slope. As the seepage increases, the wet spot spreads in size until the whole slope is wet, and the seep water slowly flows down in a sheet. Continued exposure will cause the slope to become more and more saturated and soggy until it is liable to slide or even flow out resulting in a levee failure or requiring extreme measures to prevent a failure. To prevent sloughing of the levee where the slope is steep and saturated, small V-shaped seep drains should be cut in the landside slope to remove the seepage water. These drains may be cut diagonally down the levee slope and should not be

more than 4 in. deep. Several diagonal drains may be led into one drain running straight down the levee. Horizontal drains should not be used, and extreme care should be taken not to disturb the sod unnecessarily outside of the seep drains.

The work consists of opening and clearing the various ditches so that seep water or rainwater will have a free flow from the levee into drainage ditches, which convey the water to the drainage structures through the levee. If drainage is perfected prior to high water, the effectiveness of the drainage system will be far greater than if the work is attempted after the ground has become saturated. During flood events, the gates on the drainage structures should be closed to prevent floodwaters from inundating the protected area landward of the levee. This condition may cause runoff water to pond behind the levee until the floodwaters recede. If the water behind the levee begins to cause damages, it should be pumped across the levee to the riverward side. The first drains should be cut 12 to 15 ft apart, V-shaped, no more than 4 in. deep. The drains should originate at the upper or highest limit of seepage and run straight down the slope and lead across the landside berm into a drainage system. To secure better coverage of the seeping area, additional drains spaced 4 to 6 ft should be cut between the first drains.

The above-described method of drainage is applicable to clay and other fine-grained soils on levee surfaces. It should not be used as a means of drainage on sand levees or where the foundation supporting the levee consists of sand. On sand levees, the seep drains should be omitted and the seepage allowed to *trickle* down the landside slope to the seep ditch paralleling the levee toe. If seepage through a sand levee is excessive, a blanket of clayey material should be placed on the riverside slope. If additional excavation is necessary to provide adequate drainage, the general plan described in the preceding paragraph should be followed as closely as practicable. The material excavated from the seepage ditches should be deposited on the side away from the levee, and material excavated from the off-take ditches should be deposited in such a manner that it can later be used as material for capping, if necessary. In no case should an attempt be made to cut slope drains until seepage actually appears. All traffic, animals, and personnel should be kept off seeping side slopes.

In the event of a sudden draw-down failure, loading the toe of the levee similar to the techniques described for through-seepage control can be

used. If underwater placement becomes a problem, a temporary earth-filled setback levee may be the only solution.

7.5 Sloughs and slides

Where seepage appearing high on the levee slope cannot be controlled by seep drains, and the condition grows progressively worse, there is danger that a slough or slide may develop. A slough is a condition in which the slope is excessively wet and soggy and is inclined to flow or fall away from the slope and heave or pile up at the toe. A slide is more apt to occur on steep slopes even when the soil does not appear to be extremely wet. In a slide, the slope breaks away in a clearly defined crack or cleavage plane and moves outward taking the toe of the embankment. In any case, where it appears that slope failure is likely or has occurred, the recommended treatment is reinforcement in the form of a buttress on the berm below the slide, tapering up over the failure. A brush or board mattress is always placed under the buttress and constructed in such a manner that it will permit drainage, provide a stable but flexible base for distribution of uniform pressure, bridge the failure, and anchor it against further movement.

7.6 Underseepage

Excessive underseepage can result in what is known as a sand boil. The following is a discussion of methods to treat sand boils. Piping is an extreme condition caused by excessive underseepage in which foundation materials (soil) are transported from beneath the levee. Unless corrective actions are taken, a solution channel or *pipe* may develop and enlarge to the point where the levee could fail. Early treatment of sand boils found to be transporting soil materials is the best insurance against a piping condition from developing.

The most effective method of controlling a sand boil is to reduce the head of water on the riverside of levee. This method, however, is not normally practical because it would take construction of a set-back levee to eliminate or lower the river elevation.

The most widely accepted emergency method of treating a sand boil is to construct a ring of sacked earth/sand around the boil, building up a head of water within the ring sufficient to check the velocity of flow and prevent further erosion of sand and silt. The ring should not be built to a height

that stops the flow of water because of the probability of building up an excessive local pressure head, causing additional failures and boils nearby.

The accepted method of ringing or sacking (i.e., sand-bagging) a sand boil is described as follows:

1. The base of the sack ring is prepared by clearing the adjacent ground of debris, vegetation, or other objectionable material to a width sufficient for the base of the ring. The base should then be thoroughly scarified to provide a watertight bond between the natural ground and the sack ring (a very important step).
2. The sacks are laid in a general ring around the boil, with joints staggered and with loose earth as mortar between all sacks. In general, it has been found that the best results can be obtained by commencing construction of the sack ring at its outer edge and working toward the center.
3. The ring is carried to a sufficient height to stop the flow of soil from the boil. Work is stopped when clear water only is being discharged.
4. A V-shaped drain constructed of two boards or a piece of sheet metal should be inserted near the top of the ring to carry off the water. A spillway made of sandbags can also be used to discharge water from the sandbag ring.

It is impossible to establish exact dimensions for a sack ring. Field conditions in each situation will govern. The diameter of the ring, as well as its height, depends upon the size of the boil and the flow of water from it. Field forces should determine the size of the ring upon consideration of the following:

1. The sack ring should have sufficient base width to prevent side failure. The width should be determined by the contemplated height of the ring and should be not less than 1.5 times the height.
2. The enclosed basin should be of sufficient size to permit the sacking operations to keep ahead of the flow of water. If ground weakness is indicated close to the sand boil, it is well to include the weak ground within the ring, thereby avoiding the possibility of a breakthrough later.

Sand boils at the toe of the levee are sacked in the same manner as those away from the levee, using the levee slope as one side of the enclosure. The seep drains on the levee slope should be constructed to drain the water from the sack ring. If several sand boils appear within a relatively small

radius, it is better to enclose the entire group in a sublevee or single sack ring. If sand boils break out in very low ground or deep ditches, it may be necessary to step down the head of water within the enclosure in two or three steps, by means of outside concentric rings, to avoid a *blowout* near the ring.

An inverted filter is an expedient and economical means to control excessive seepage such as sand boils. A fine sand and/or filter fabric is normally placed over the seepage area with successively larger granular material placed on top. The section will allow the seepage water to be safely removed while holding down or trapping the fine soil material, preventing the development of a piping situation. An alternate method of ringing sand boils is by the use of corrugated sheet-steel piling or pipe culverts. Using sheet-steel piling or pipe accomplishes the same task faster than sandbagging but is limited in use by the availability of material, equipment, and location of boils. However, circumstances will dictate the system or method most applicable.

There are generally two methods used to control levee failure caused by water flowing through holes in the levee created by burrowing animals: ring the landside opening with sacks the same as for a sand boil and plug the opening on the riverside with sandbags or plastic sheeting. When a leaking burrow is first observed, effort should be made to first stop the flow from the riverside by spreading a tarpaulin or plastic sheeting on the riverside slope and weighting it down with sandbags. A single sack over the riverside opening of the burrow may stop the burrow from leaking if the opening can be found, but the tarpaulin or plastic sheeting has the advantage of covering a larger area since the intake opening might not necessarily be exactly opposite the discharge opening. The tarpaulin or plastic sheet would probably be more impervious than a sandbag and would therefore provide a better seal.

If the burrow hole is high up on the landside slope with minimal hydraulic head, sacks tamped directly into the outlet will effectively stop the flow. It would be necessary to cut a small notch or bench at the opening to seat the sacks into place. Landside treatment, which may be required if the riverside opening cannot immediately be stopped, is to build a sack ring similar to a boil ring around the landside opening with a sufficient base width to support a ring to a height sufficient to stop the flow of water. This ring differs from a boil ring in that it is required to stop the flow of water.

The time, material, and labor required for a ring emphasizes the importance of first attempting to stop the flow from the riverside of the levee structure.

7.7 Levee breaks

Where it is practical and desirable to do so, closure of a break in a levee will reduce the period of inundation of the property inside of the levee, prevent the break from widening, and reduce the damage caused by subsequent rises that may occur before the levee can be repaired. Generally, a break closure should not be attempted on a rising river stage or on an extremely high stage. Conditions could develop such that it would become impossible to accomplish the closure. The time to attempt a closure is on a falling river stage when the velocity and turbulence of the flow through the break has decreased sufficiently to assure complete success of the effort.

There are undoubtedly several acceptable methods of making a closure. However, each closure must be considered as a special case depending on the general location, size, river stage, economics, and the health and safety of the general public. Seepage through and under a levee may be controlled to prevent a levee failure from occurring; however, a significant quantity of water may pond on the landward side of the levee with no place to drain to. In this situation, pumping may be used to prevent damages caused by seep water.

One levee closure plan, which has been developed and successfully used by the Corps of Engineers, is detailed in the following paragraphs. It should be considered for use only under specific situations where the plan and general conditions are complementary and not as a standard procedure for all closures.

The structure is composed of two parts: a timber trestle filled with sandbags to shut off the free flow of water and an earth-filled mud box landward of the trestle to reinforce and make the structure watertight. A scour hole usually forms in the break slightly landward and enlarges to the landside. The closure structure should be located far enough away from the edge to allow for enlargement of the scour hole, and the structure may be placed either on the landside or the riverside of the crevasse depending on which has the shallower water and the least amount of obstructions. The ends of the structure should join the existing levee well back from the

edges of the break to allow for caving while the closure is being built. Trees should be cut off just above the water surface to prevent any movement of sandbags caused by trees swaying in the wind. The closure should never be started until all required labor and material are available at the site so that closure can be made without interruption. The delay of a few minutes at a critical time may mean the loss of the closure.

Closing a levee break entails considerable danger to personnel working on the closure. Handrails should be installed where needed, the project should be well lighted, and employees should wear life vests when working near water. At least two boats, equipped with oars and ring buoys with handlines, and manned at all times by experienced operators, should be anchored just below the levee break. An experienced first-aid team equipped with first-aid equipment should be available at all times. In areas where soil material and earthmoving equipment are available, the levee closure can be constructed of earth.

8 Other Considerations

The topics that follow are general considerations that apply to all flood risk reduction projects. The reliability of flood protection can change if external variables change (such as climate and land use change increasing the DBFL), design flaws exist (such as inadequate stormproofing of pumping stations), or insufficient maintenance is performed (such as failing to inspect and repair all aspects of the flood-protection system).

8.1 Climate change

Climate change should be addressed in the design and evaluation of flood risk reduction projects. Recent USACE guidance addresses the incorporation of sea-level change into project planning and designs (EC 1165-2-212). The recently completed *New Orleans District, Hurricane and Storm Damage Risk Reduction System* (USACE 2013) incorporated sea-level rise into the design of the levee system (along with subsidence and settlement and other factors). Changes in flood magnitudes and frequencies are discussed in *Design-Basis Flood Estimation for Site Characterization at Nuclear Plants in the United States of America* (NRC 2011).

8.2 Large storm event resiliency

Flood risk reduction systems need to perform during large storm events. This means that every aspect of system performance must be considered in advance. Operational procedures that work well under smaller or localized flood events may not work at all when events are larger and cover an entire region. Hurricane events can be used as illustrations. Utilities such as power, phone, and internet service may be unavailable for days or weeks. Gasoline may not be available. Highways, rail lines, and public transportation may be unusable due to debris, flooding, landslides, or other reasons. It may be impossible to get key personnel, equipment, and supplies on site. In short, sources of assistance that are often taken for granted may be totally unavailable. The Greater New Orleans Hurricane and Storm Damage Risk Reduction System (http://www.mvn.usace.army.mil/Missions/HSDRRS.aspx) was built with multiple features to ensure continued operation during future storm events. For example, the pump stations have been stormproofed with backup power and fuel sources. Critical electronic equipment has been raised to avoid submersion. Safe rooms have been provided and strengthened to withstand

hurricane-force winds. Emergency food and water are located on-site so that station operators can stay on the job during a storm event. In some cases, the stormproofed area in the pump station includes windows so that operators can view gate operation and water levels from inside during the storm event. Preparation for large storm events must consider a lack of services and supplies on a regional scale, for an extended time period.

8.3 Inspection and evaluation

Flood risk reduction systems require periodic inspection, maintenance, rehabilitation, and repair. Many existing levee systems have serious deficiencies and may fail or require heroic floodfighting measures during a flood event. The level of protection should be re-evaluated periodically, since hydrologic and hydraulic conditions may change over time. These changes often increase flood risk. For example, changes in land use may cause increased peak flows. Additional vegetation or structures in the floodplain may cause higher stages and increase the slope of the water-surface profile, leading to an increased risk of seepage and overtopping. Conditions downstream of the project may increase flood levels or erosion. Sedimentation (deposition in the channel or floodplain) may cause increases in stage for either the entire project or in a localized area. These conditions may cause overtopping at events smaller than the design event, or they may cause overtopping at an unplanned location. Channel migration or channel incision may erode earthen embankments, with the potential to cause breaches during floods. Settlement and subsidence may lower the top of protection, and many factors may change the water-surface profile. Sites that were evaluated as *dry* during the planning phase of a project may be subject to flooding under changed conditions at a later date. In summary, projects are not static. Both the project components and activities in the watershed upstream and downstream may change over time. These changes are seldom positive from a flood risk reduction standpoint and may diminish the reliability of the project. Documentation pertaining to the inspection of NRC-licensed activities can be found at http://www.nrc.gov/reading-rm/doc-collections/insp-manual/.

9 Summary and Recommendations

Flood protection methods for nuclear power plants fall into one of the following five categories:

- dry sites
- exterior (primary) barriers
- incorporated (secondary) barriers
- temporary barriers
- interior drainage to accommodate locally intense precipitation.

Dry sites are located above the DBFL. The DBFL is the maximum water elevation attained by the controlling flood, including coincident wind-generated wave effects. At a dry site, because a site is above the DBFL, all safety-related structures, systems, and components are not affected by external flooding but are subject to flooding from local intense precipitation. Exterior barriers are natural or engineered structures exterior to the immediate site. Examples of exterior barriers include earthen embankments, sea walls, floodwalls, revetments, and breakwaters. When properly designed and maintained, exterior barriers can produce a site with the flood risk approaching that of a dry site. Incorporated barriers are engineered structures located at the nuclear power plant site/environment interface. Examples of incorporated structures include waterproof walls and sealed hatches.

The USACE recommends multiple layers of proven exterior structural barriers for flood protection. Currently, adequate data and analyses do not exist in order for the USACE to recommend the use of incorporated or temporary barriers as part of a complete flood-protection system. The reliability of incorporated and temporary systems is insufficient. However, incorporated and temporary barriers may be used to supplement a complete flood-protection system that includes a properly designed and maintained external barrier, an internal drainage system, and redundant pumping stations.

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14. ABSTRACT
Current regulatory guidance on flood protection at nuclear power plants is contained in the Regulatory Guide 1.102, “Flood Protection for Nuclear Power Plants” (Nuclear Regulatory Commission, 1976). Regulatory Guide 1.102 requires that structures, systems, and components important to safety be designed to withstand the effects of natural phenomena such as floods, tsunami, and seiches without loss of capability to perform their safety functions. These requirements remain the same in 2013 as they did in 1976, however, the current technology, understanding and practice has changed. This report describes current (2013) flood protection feature types and applications for protecting safety-related structures, systems, and components. This document compiles and ties together information from multiple U.S. Army Corps of Engineers manuals in order to provide a resource of scientific and technical background concerning various flood protection structures and their associated risk and reliability. Several chapters and sections in this report summarize foundational U.S. Army Corps of Engineers documents.

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